INTERNATIONAL STANDARD

ISO 19902

First edition
2007-12-01

Petroleum and natural gas industries — Fixed steel offshore structures

Industries du pétrole et du gaz naturel — Structures en mer fixes en acier

Reference number
ISO 19902:2007(E)

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Foreword

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

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

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

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

ISO 19902 was prepared by Technical Committee ISO/TC 67, *Materials, equipment and offshore structures for petroleum, petrochemical and natural gas industries*, Subcommittee SC 7, *Offshore structures*.

This first edition of ISO 19902 cancels and replaces ISO 13819-2:1995, which has been technically revised.

ISO 19902 is one of a series of standards for offshore structures. The full series consists of the following International Standards:

— ISO 19900, *Petroleum and natural gas industries — General requirements for offshore structures*

— ISO 19901-1, *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 19901-3, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 3: Topsides structure*[1)]

— ISO 19901-4, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 4: Geotechnical and foundation design considerations*

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

— ISO 19901-6, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 6: Marine operations*[2)]

— ISO 19901-7, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units*

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

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1) Under preparation.

2) To be published.

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

— ISO 19904-1, *Petroleum and natural gas industries — Floating offshore structures — Part 1: Monohulls, semi-submersibles and spars*

— ISO 19904-2, *Petroleum and natural gas industries — Floating offshore structures — Part 2: Tension leg platforms*[3)]

— ISO 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups*[3)]

— ISO/TR 19905-2, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 2: Jack-ups commentary*[3)]

— ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*[3)]

3) Under preparation.

Introduction

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

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

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

Annex A provides background to and guidance on the use of this document and needs to be read in conjunction with the main body of this document. The clause numbering in Annex A is the same as in the normative text to facilitate cross-referencing.

Materials, welding and weld inspection requirements can be based either on a “material category” or on a “design class” approach, as discussed in Clauses 19 and 20. If the material category approach is used, the corresponding provisions of Annexes C and E are normative; if the design class approach is used, the corresponding provisions of Annexes D and F are normative.

Annex G gives requirements on fabrication tolerances.

Regional information on the application of the document to certain specific offshore areas is provided in informative Annex H.

To meet certain needs of industry for linking software to specific elements in this International Standard, a special numbering system has been permitted for figures, tables, equations and bibliographic references.

Petroleum and natural gas industries — Fixed steel offshore structures

1 Scope

This International Standard specifies requirements and provides recommendations applicable to the following types of fixed steel offshore structures for the petroleum and natural gas industries:

— caissons, free-standing and braced;

— jackets;

— monotowers;

— towers.

In addition, it is applicable to compliant bottom founded structures, steel gravity structures, jack-ups, other bottom founded structures and other structures related to offshore structures (such as underwater oil storage tanks, bridges and connecting structures), to the extent to which its requirements are relevant.

This International Standard contains requirements for planning and engineering of the following tasks:

a) design, fabrication, transportation and installation of new structures as well as their future removal;

b) in-service inspection and integrity management of both new and existing structures;

c) assessment of existing structures;

d) evaluation of structures for reuse at different locations.

NOTE 1 Specific additional requirements for the design of fixed steel offshore structures in arctic environments are to be contained in ISO 19906[1].

NOTE 2 Requirements for topsides structures are to be contained in ISO 19901-3[2], for marine operations in ISO 19901-6[3] and for the site-specific assessment of jack-ups in ISO 19905-1[4].

2 Normative references

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

ISO 10414-1, *Petroleum and natural gas industries — Field testing of drilling fluids — Part 1: Water-based fluids*

ISO 12135, *Metallic materials — Unified method of test for the determination of quasistatic fracture toughness*

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

ISO 19900:2002, *Petroleum and natural gas industries — General requirements for offshore structures*

ISO 19901-1:2005, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 1: Metocean design and operating considerations*

ISO 19901-2, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 2: Seismic design procedures and criteria*

ISO 19901-4, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 4: Geotechnical and foundation design considerations*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 19900, ISO 19901-1, ISO 19901-2 and ISO 19901-4, and the following apply.

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

NOTE 1 Abnormal design situations are used to provide robustness against events with a probability of exceedance of typically between 10^{-3} and 10^{-4} per annum by avoiding, for example, gross overloading.

NOTE 2 Abnormal values and events have probabilities of exceedance of the order of 10^{-3} to 10^{-4} per annum. In the limit state checks, some or all of the partial factors are set to 1,0.

NOTE 3 Adapted from ISO 19901-1:2005, definition 3.1.

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

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

3.3
after damage design situation
design situation for which the condition of the structure reflects damage due to an accidental design situation and for which the environmental conditions are specially defined

3.4
analysis type
method including governing equations for deriving action effects

EXAMPLE Static analysis, transient dynamic analysis, non-linear analysis.

3.5
basic variable
one of a specified set of variables representing physical quantities which characterize actions, environmental influences, geometrical quantities, or material properties including soil properties

[ISO 19900:2002]

3.6
boundary conditions
actions and constraints on a [section of a] structural component [or a group of structural components] by other structural components or by the environment surrounding it

NOTE Boundary conditions can be used to generate reaction forces at locations of restraint.

3.7
braced caisson
monotower where the lower part of the monocolumn is supported laterally by one or more inclined braces between the column and one or more foundation piles

3.8
bucket foundation
foundation consisting of a cylindrical shell open on one end and installed by suction

3.9
characteristic value
value assigned to a basic variable associated with a prescribed probability of not being violated by unfavourable values during some reference period

NOTE The characteristic value is the main representative value. In some design situations a variable can have two characteristic values, an upper and a lower value.

[ISO 19900:2002]

3.10
compliant bottom founded structure
structure which is supported at its base by foundation piles or by another non-superficial foundation system and which is sufficiently flexible that applied lateral dynamic actions are substantially balanced by inertial reactions

NOTE Although this International Standard is applicable to fixed steel offshore structures and is not intended to form a complete standard for compliant bottom founded structures, some of its requirements and guidance can be applied to compliant bottom founded structures. Parts of its requirements can also apply to other bottom founded structures (e.g. some forms of minimal structure), as considered appropriate and as agreed upon by parties concerned on a case by case basis.

3.11
consequence category
classification system for identifying the environmental, economic and indirect personnel safety consequences of failure of a platform

NOTE Categories for environmental and economic consequences are (see 6.6.3)

C1 high environmental and/or economic consequences,

C2 medium environmental and/or economic consequences, and

C3 low environmental and economic consequences.

3.12
critical component
structural component, failure of which would cause failure of the whole structure, or a significant part of it

NOTE A critical component is part of the primary structure.

3.13
deformation capacity
ability of a structure or structural component to deform without significant loss of resistance, or the extent to which it can do so

3.14
design service life
assumed period for which a structure is to be used for its intended purpose with anticipated maintenance, but without substantial repair being necessary

[ISO 19900:2002]

3.15
design value
value derived from the representative value for use in the design verification procedure

[ISO 19900:2002]

3.16
dynamic amplification factor
DAF
ratio of a dynamic action effect to the corresponding static action effect

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

3.17
explosion
rapid chemical reaction of gas or dust in air

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

3.18
exposure level
classification system used to define the requirements for a structure based on consideration of life-safety and of environmental and economic consequences of failure

[ISO 19900:2002]

3.19
extreme value
value of a parameter used in ultimate limit state checks, in which a structure's global behaviour is intended to stay in the elastic range

NOTE Extreme values and events have probabilities of exceedance of the order of 10^{-2} per annum.

[ISO 19901-1:2002]

3.20
fit-for-purpose
meeting the intent of an International Standard although not meeting specific provisions of that International Standard in local areas, such that failure in these areas will not cause unacceptable risk to life-safety or the environment

[ISO 19900:2002]

3.21
fixed structure
structure that is bottom founded and transfers all actions on it to the sea floor

[ISO 19900:2002]

3.22
free-standing caisson
monotower where the structure consists, over its full height, of a single vertical column that continues into the seabed as the foundation pile

3.23
global analysis
determination of a consistent set of internal forces and moments, or stresses, in a structure that are in equilibrium with a defined set of actions on the entire structure

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

3.24
hazard
potential for human injury, damage to the environment, damage to property, or a combination of these

[ISO 13702:1999]

NOTE Several of the usual hazards to a platform (e.g. extreme storms) are treated as design situations for a structure. In this International Standard, hazards are errors and abnormal and accidental situations.

3.25
jacket
fixed structure with leg piles and axial force transfer from the structure and topsides into the piles at the top of the structure

NOTE See 6.1.2.

3.26
jack-up
mobile offshore unit that can be relocated and is bottom founded in its operating mode

[ISO 19900:2002].

NOTE See 6.1.4.

3.27
life-safety category
classification system for identifying the applicable level of life-safety for a platform

NOTE Categories for life-safety are (see 6.6.2)

S1 manned non-evacuated,

S2 manned evacuated, and

S3 unmanned.

3.28
load arrangement
identification of the position, magnitude and direction of a free action

3.29
load case
compatible load arrangements, sets of deformations and imperfections considered simultaneously with permanent actions and fixed variable actions for a particular design or verification

3.30
local analysis
determination of a consistent set of internal forces and moments, or stresses, in a cross-section of a structural component, or in a subset of structural components forming part of the structural system, that are in equilibrium with the boundary conditions

3.31
maintenance
set of activities performed during the working life of the structure in order to enable it to fulfil the requirements for reliability

NOTE Activities to restore the structure after an abnormal, accidental or seismic event are outside the scope of maintenance.

3.32
monotower
fixed structure in which the whole structure, or at least the upper part of the structure, consists of a single vertical column (tubular or framed) that carries the topsides

NOTE Where only the upper part of the structure is a single vertical column, the lower part of the structure consists of tubular members or frames that connect the vertical column to the foundation piles or to another non-superficial foundation system that supports the monotower at its base, such as bucket foundations.

3.33
nominal geometrical properties
properties of a structural component derived from its representative geometrical dimensions, remote from the location under consideration and with any corrosion allowance removed

NOTE 1 The nominal cross-section of a member at a joint is the cross-section beyond any joint can or brace stub forming part of the joint.

NOTE 2 Nominal geometrical properties are used in a global analysis to calculate the global behaviour of the structure.

NOTE 3 The nominal thickness of a component excludes any thickening of the component due to weldments.

3.34
nominal stress
stress calculated in a sectional area, including the stress raising effects of the macro-geometrical shape of the component of which the section forms a part, but disregarding the local stress raising effects from the section shape and any weldment or other fixing detail

NOTE Overall elastic behaviour is assumed when calculating nominal stresses.

3.35
nominal value
value assigned to a basic variable determined on a non-statistical basis, typically from acquired experience or physical conditions

[ISO 19900:2002]

3.36
owner
representative of the companies which own a development

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

3.37
primary structure
all main structural components that provide the structure's main strength and stiffness

3.38
quasi-static analysis
static analysis of a structure subjected to actions that vary slowly in relation to the structure's fundamental natural period such that the influence of structural accelerations can be either safely neglected or is approximated by using an equivalent quasi-static action

3.39
redundancy
ability of a structure to find alternative load paths following failure of one or more non-critical components, thus limiting the consequences of such failures

NOTE All structures having redundancy are statically indeterminate.

3.40
regulator
authority established by a national governmental administration to oversee the activities of the offshore oil and natural gas industries within its jurisdiction, with respect to the overall safety to life and protection of the environment

NOTE 1 The term *regulator* can encompass more than one agency in any particular territorial waters.

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

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

3.41
repair
activities performed to preserve or to restore the function of a structure that fall outside the definition of maintenance

3.42
representative value
value assigned to a basic variable for verification of a limit state

[ISO 19900:2002]

3.43
representative yield strength
stress at which a material exhibits a specified deviation from proportionality of stress and strain

NOTE An offset of 0,2 % is used for many metals, including the steels covered by this International Standard.

3.44
reserve strength ratio
measure of the capacity of a structural system to withstand overload

NOTE For fixed steel offshore structures, the concept of a reserve strength ratio is usually applied to environmental actions (see 7.10), but is not limited to these actions.

3.45
risk reducing measures
measures undertaken to reduce either the probability or the consequences of an accident, including the preparation of contingency plans

3.46
robustness
ability of a structure to withstand events with a reasonable likelihood of occurring without being damaged to an extent disproportionate to the cause

3.47
secondary structure
structural components that, when removed, do not significantly alter the overall strength and stiffness of the structure

3.48
steel gravity structure
fixed structure that is held in place against environmental actions solely by the weight of the structure and any contained ballast, together with foundation resistance resulting from its weight and lateral resistance from any skirts

NOTE Although this International Standard is not intended to form a complete standard for gravity structures, some of the requirements and guidance could be applied to gravity structures.

3.49
strength
mechanical property of a material indicating its ability to resist actions, usually given in units of stress

3.50
stress concentration factor
SCF
factor relating a nominal stress to the local stress at a detail

NOTE SCFs are very important for and especially used in determining fatigue damage (see 16.10.2).

3.51
structural analysis
process or algorithm for determining action effects from a given set of actions

NOTE Structural analyses are performed at three levels (global analysis of an entire structure, local analysis of a structural component, local analysis of a section of a structural component) using different structural models.

3.52
structural component
physically distinguishable part of a structure

[ISO 19900:2002]

NOTE The main structural components of a fixed steel structure are tubular members (see Clause 13), tubular joints (see Clause 14), mechanical connectors and clamps (see Clause 15), foundation piles and bucket foundations (see Clause 17).

3.53
structural system
load-bearing components of a structure and the way in which these components function together

[ISO 19900:2002]

3.54
topsides
structures and equipment placed on a supporting structure (fixed or floating) to provide some or all of a platform's functions

[ISO 19900:2002]

NOTE 1 For a ship-shaped floating structure, neither the deck nor a superstructure integral with the hull is part of the topsides.

NOTE 2 For a jack-up, the hull is not part of the topsides.

NOTE 3 A separate fabricated deck or module support frame is part of the topsides.

3.55
tower
fixed structure that is supported by foundation arrangements at the base of the structure

NOTE See 6.1.3.

3.56
utilization
member utilization
joint utilization
maximum value of the ratio of the generalized representation of the design stress (force) in a structural component to the generalized representation of the design resistance in stress (force) units of the component

NOTE 1 The utilization is the maximum absolute value of the ratio for all conditions being considered.

NOTE 2 Only utilizations smaller than or equal to 1,0 satisfy the design criteria for a particular limit state.

NOTE 3 The design stress (force) is the stress (force) due to factored actions: $F_d = F_r \gamma_f$. The design resistance is the representative resistance divided by the partial resistance factor: $R_d = R_r/\gamma_R$.

NOTE 4 For members and joints subjected to a single force, the utilization, U, is equal to: $F_d/R_d = F_r \gamma_f \gamma_R/R_r$.

NOTE 5 For members and joints subjected to combined forces, the internal stress (force) pattern and the design resistance combine into an interaction equation. If the interaction equation governing the design check is, or can be, reduced to an inequality of the form $I \leqslant 1{,}0$, then the utilization is equal to I.

4 Symbols

The following is a summary of the main symbols that are used throughout this International Standard. Many other symbols are locally defined where they are used. Local use includes main symbols with one or more subscripts when a more specific use and associated definition of the symbol is intended.

A	accidental action (from ISO 19900)
A	area
C	coefficient in general
C_d	hydrodynamic drag coefficient
C_m	hydrodynamic inertia coefficient
C_m	moment reduction factor
C_r	seismic reserve capacity factor
C_s	shape coefficient
C_s	slamming coefficient
c_u	undrained shear strength of the soil, in stress units
D	diameter of a circular cylindrical element
D	diameter of a tubular joint chord
D	Palmgren-Miner fatigue damage ratio
D	equivalent quasi-static action representing dynamic effects
D_r	relative density of sand
d	water depth
d	diameter of tubular joint brace
E	Young’s modulus of elasticity, which may be taken as 205 000 MPa for structural steel, in lieu of more detailed information
E	quasi-static environmental action
E_e	extreme quasi-static environmental action due to wind, waves and current
E_o	environmental action due to owner defined operating wind, wave and current parameters

F action

F_d design value of action (from ISO 19900)

F_r representative value of action (from ISO 19900)

f unit skin friction of a pile, in stress units

f representative strength in general, in stress units

f_b representative bending strength, in stress units

f_c representative axial compressive strength, in stress units

f_{cu} unconfined cube strength of grout

f_e Euler buckling strength, in stress units

f_h representative hoop buckling strength, in stress units

f_t representative axial tensile strength, in stress units

f_v representative shear strength, in stress units

f_y representative yield strength, in stress units

G permanent action (from ISO 19900)

g acceleration due to gravity

g gap

H height of an individual wave

H_s significant wave height

I moment of inertia

K factor (locally defined with subscript for specific meaning)

K effective length factor

K dimensionless coefficient of lateral earth pressure

K Keulegan-Carpenter number

k factor (locally defined with subscript for specific meaning)

k wave number

k_{DAF} dynamic amplification factor

L length

M moment (bending or torsion) due to factored actions

M_d design value of joint bending moment strength, in moment units

M_{uj} representative value of joint bending moment strength, in moment units

m inverse slope of the S-N curve

m mass

N number of cycles to failure in fatigue of a specified constant amplitude stress range, S

n number of occurrences of a specified stress range, S, during a specified time

P percentage

P member axial force due to factored actions

P_d design value of joint axial strength, in force units

P_{uj} representative value of joint axial strength, in force units

P_f probability of failure

$P\text{-}\Delta$ second order bending effect due to axial compressive force, P, and lateral deflection, Δ

$P(x)$ cumulative probability of variable x

p factored hydrostatic pressure

p_a atmospheric pressure

p_0' effective overburden stress of the soil

$p(x)$ probability density function of variable x

$p\text{–}y$ lateral soil resistance versus local pile displacement

Q variable action (from ISO 19900)

Q_d design axial pile capacity

Q_f representative value of the total skin friction resistance of a pile, in force units

Q_f chord force factor

Q_g gap factor for a joint

Q_p representative value of the end bearing capacity of a pile, in force units

Q_r representative value of the axial pile capacity

Q_u chord strength factor

$Q\text{–}z$ pile end bearing resistance versus pile tip displacement

Q_β geometrical factor for a joint

q unit end bearing of a pile, in stress units

R radius of a circular cross-section

R_d design value of a component resistance (from ISO 19900)

Re Reynolds number

R_{RS} reserve strength ratio (RSR)

r radius of gyration

S internal force (action effect)

S constant amplitude stress range for fatigue assessments

Sr Strouhal number

S_r Stress range

T quasi-static transient action

T wall thickness of the chord member

T period in general

T_i intrinsic wave period

T_z average zero-crossing wave period

t time

t wall thickness in general

t wall thickness of the brace member

$t\text{–}z$ soil pile shear transfer versus local pile displacement

U utilization

V beam shear due to factored actions

V displaced volume

V coefficient of variation

Z_e elastic section modulus

Z_p plastic section modulus

α slope angle of a cone section

β ratio of brace outside diameter to chord outside diameter

γ ratio of chord outside radius to chord wall thickness

γ' submerged unit weight of soil

γ_f partial action factor of which the value reflects the uncertainty or randomness of the action (from ISO 19900)

γ_R partial resistance factor of which the value reflects the uncertainty or variability of the component resistance including those of material properties (from ISO 19900)

λ wave length

λ column slenderness parameter

ν Poisson's ratio, which may be taken as 0,3 for structural steel, in lieu of more detailed information

υ kinematic viscosity of water

θ angle in general

θ_m mean wave direction

ρ_a mass density of air, which may be taken as 1,22 kg/m^3 in lieu of more detailed information

ρ_s mass density of steel, which may be taken as 7 850 kg/m^3 for structural steel, in lieu of more detailed information

ρ_w mass density of sea water, which may be taken as 1 025 kg/m^3, in lieu of more detailed information

σ stress in general

τ shear stress in general

τ ratio of brace wall thickness to chord wall thickness

ϕ' angle of internal friction in sand

Ω ratio of frequency of excitation to natural frequency

ω circular frequency

ω_e frequency of excitation

ω_n natural frequency in general

ω_1 first natural frequency

ξ factor (locally defined with subscript for specific meaning)

ξ fraction of critical damping

$\varnothing$ diameter

5 Abbreviated terms

ADS	atmospheric diving suit
ALE	abnormal level earthquake
ALS	accidental limit states
API	American Petroleum Institute
ASD	allowable stress design
AW	as-welded (in Annex D)
CHS	circular hollow section
COV	coefficient of variation
CTOD	crack tip opening displacement
CVN	Charpy V-notch
DAF	dynamic amplification factor
DC	design class
ELE	extreme level earthquake
ERW	electric resistance welded
FEA	finite element analysis
FLS	fatigue limit states
FMD	flooded member detection
GS	geometrical stress
GSR	geometrical stress range
HAT	highest astronomical tide
HAZ	heat affected zone
HISC	hydrogen induced stress cracking
IACS	International Association of Classification Societies
IC	impressed current
ipb	in-plane bending
JIP	joint industry project
LAST	lowest anticipated service temperature
LAT	lowest astronomical tide
LBZ	local brittle zone
LRFD	load and resistance factor design
MC	material category
MIC	microbiologically influenced corrosion
MPI	magnetic particle inspection
MPM	most probable maximum

NDT	non-destructive testing
opb	out-of-plane bending
PLS	progressive collapse limit state
POD	probability of detection
PQR	procedure qualification record
PWHT	post weld heat treatment
QA	quality assurance
QC	quality control
QMS	quality management system
Q&T	quenched and tempered
RAO	response amplitude operator
RHS	rectangular hollow section
rms	root-mean-square
RSR	reserve strength ratio
RT	radiographic testing
SAW	submerged arc welding
SCF	stress concentration factor
SDOF	single degree of freedom
SIM	structural integrity management (system)
SLS	serviceability limit states
SMYS	specified minimum yield strength
SRA	structural reliability analysis
SRC	seismic risk category
SRD	soil resistance to driving
SSSV	subsurface safety valve
TMCP	thermo-mechanically controlled process
TRF	transfer function
ULS	ultimate limit states
UT	ultrasonic testing
UU	unconsolidated and undrained
VIV	vortex induced vibrations
WPS	welding procedure specification
WSD	working stress design
WPQ	welding procedure qualification

6 Overall considerations

6.1 Types of fixed steel offshore structure

6.1.1 General

A fixed steel offshore structure can take many forms including:

— braced caisson (3.7);

— free-standing caisson (3.22);

— jacket (3.25, see 6.1.2);

— monotower (3.32).

— tower (3.55, see 6.1.3);

— steel gravity structure (3.48);

— jack-up (3.26, see 6.1.4).

This International Standard specifies requirements for braced caissons, free-standing caissons, jackets, monotowers and towers. Some of the requirements may also be used, as appropriate, for other forms of steel bottom founded offshore structures. The design of other forms of fixed steel bottom founded offshore structures shall achieve a level of reliability consistent with that intended by this International Standard.

NOTE There have been historical differences in the usage and understanding of the terms “jacket” and “tower”, particularly between the USA and Europe. The difference in such understanding of the terms has no significant impact on the application of this International Standard as long as the differences in structural behaviour are considered in the analyses of the different structures.

6.1.2 Jackets

Jackets and towers are superficially similar structures, the essential differences between them being the arrangement of the foundation piles and, associated with this, the location of axial force transfer from the structure and topsides into the piles. The location and method of transferring the axial forces results in significant differences in the behaviour of jackets and towers.

A jacket is a welded tubular space frame with three or more vertical (or near vertical) tubular chords (the legs) and a bracing system between the legs. The jacket provides support for the foundation piles, conductors, risers, and other appurtenances.

A jacket foundation includes leg piles which are inserted through the legs and are connected to the legs either at the top, by welding or mechanical means, or along the length of the legs, by grouting. Additional piles, called skirt piles, can be inserted through and connected to sleeves at the base of the structure. Leg and skirt piles jointly anchor the structure and transfer both vertical and horizontal actions to the seabed.

Where the foundation piles are connected to the legs only at the top of the jacket, the axial forces (including moments caused by shear forces) are transferred to the piles at the connection and the jacket “hangs” from the piles.

Where the foundation piles are connected to the legs by grouting along the full length of the leg, the jacket behaviour is similar to that of a tower, with the legs and piles acting together as composite components.

6.1.3 Towers

A tower type structure is supported by foundation arrangements at the base of the structure.

The tower is again made of a welded tubular space frame with three or more vertical (or near vertical) tubular chords, called legs, with a bracing system between the legs. The tower provides support for the topsides, conductors, risers and other appurtenances.

A tower foundation usually includes cluster piles which are inserted through and connected to sleeves around the (corner) legs at the base of the structure. Additional piles, called skirt piles, can be inserted through and connected to sleeves at the base and along the perimeter of the structure. As an alternative to piles, a tower can be supported by another non-superficial foundation system that supports it at its base, such as bucket foundations. Cluster and skirt piles, or another non-superficial foundation system, anchor the structure and transfer both vertical and horizontal actions to the seabed.

The global behaviour of a tower is that of a vertical cantilever, with all actions being transferred to the foundation system at the base of the tower.

6.1.4 Jack-ups

A jack-up comprises a floating hull and one or more legs, which can move up and down relative to the hull. A jack-up reaches its operational mode by lowering the leg(s) to the sea floor and then raising the hull to the required elevation. The majority of jack-ups have three or more legs, each of which can be moved independently and which are supported on the sea floor by spudcans.

The majority of jack-ups are built for short-term operations at different locations around the world. As metocean and foundation conditions vary between locations, such jack-ups are assessed for each particular location, see ISO 19905-1[4]. A few jack-ups are purpose built for production operations at a single location, albeit that there can be an intent for eventual reuse at further locations.

Two options are available for a jack-up within the framework of the ISO 19900 series, these are

a) design, assessment and operation to the requirements of this International Standard, and

b) site-specific assessment to the requirements of ISO 19905-1[4], in which case either the periodic inspection requirements of the IACS member or the alternative requirements for long-term deployment in ISO 19905-1[4] shall be satisfied.

NOTE ISO 19905-1[4] applies only to structures holding valid certification as self-elevating units from an IACS member, or equivalent.

Option a) above shall be used for a jack-up purpose built for production at a particular location and where option b) does not apply. Option b) should be used for a jack-up built to operate in a variety of locations. In other cases, when an existing jack-up is to be used for an extended period at a single location for production (rather than for exploration, appraisal or other short term application) the owners and regulators shall apply either the reuse requirements (see Clause 25) of this International Standard or the requirements of ISO 19905-1[4].

When option a) is selected, some aspects of the requirements of this International Standard do not apply, including the determination of environmental actions and action effects due to dynamic response, structural behaviour and foundation behaviour. The alternative procedures given in ISO 19905-1[4] may be used in such cases; however, the procedures shall be modified as necessary to give an equivalent level of structural integrity to that intended by this International Standard. The foundation strength, the foundation preload, overturning, leg chord strength and the strength of the holding system shall be checked in accordance with the requirements of ISO 19905-1[4].

A limitation on the use of this International Standard for jack-ups is the strength and ductility of high strength jack-up steels (with yield strengths of 700 N/mm^2 or more) for major structural components — as compared with yield strengths of less than 500 N/mm^2 for other conventional fixed steel offshore structures. The designer or assessor of such jack-up units shall take note of the limitations of this International Standard for steels with yield strengths higher than 500 N/mm^2 and shall obtain sufficient data to justify the design for such cases.

6.2 Planning

6.2.1 General

Adequate planning shall be completed before actual design is started in order to obtain a safe, workable, and economical offshore structure. The initial planning shall include the determination of all limit states, design situations and design criteria upon which the design of the structure is to be based, following the general requirements and conditions specified in ISO 19900.

6.2.2 Foundations and active geological processes

The capacity and suitability of the foundations and the susceptibility to active geological processes shall be considered in selecting the type of structure and the location. Requirements and guidance are given in Clause 17 and in ISO 19901-4.

6.2.3 Design for inspection and maintenance

ISO 19900 notes that “Structural integrity, serviceability throughout the intended service life, and durability are not simply functions of the design calculations but are also dependent on the quality control exercised in construction, the supervision on site, and the manner in which the structure is used and maintained.” Therefore, during the planning stage a philosophy for inspection and maintenance should be developed. The design of the structure as a whole, as well as the structural details, should be consistent with this philosophy. In preparing the philosophy, a realistic assessment should be made of the ability to actually achieve the intended quality of the results of inspection and maintenance efforts. This assessment should be taken into account when progressing the design and fine-tuning design situations and criteria. Relevant requirements related to inspection and maintenance requirements are given in Clauses 21 and 23.

6.2.4 Design situations and criteria

Design situations as used herein include all the service and operational requirements resulting from the intended use of the structure and the environmental conditions that can affect the design of the structure in accordance with ISO 19900. In the absence of site-specific information on the environmental conditions, ISO 19901-1 contains indicative values for the extreme conditions in certain areas. These values may be used for conceptual design studies. The owner shall review the validity of these values and, if necessary, use site-specific data for the final design of a structure, see also 6.5.2 and 7.3. Criteria that are to be met by the design are directly related to the specific formulation of the design situations. Therefore, design situations and design criteria shall not be separated from one another. They are jointly specified in Clauses 8 to 17.

6.2.5 Regulations

The requirements of national regulations and standards where a structure will be used can be different from those given in this International Standard. It shall be ensured that the requirements of safety, reliability and durability implicit in the requirements of this International Standard are met. This applies to all phases of planning, design, fabrication, transportation, installation, service and removal.

6.3 Service and operational considerations

6.3.1 General considerations

The principal service and operational requirements to be considered to establish the design basis for fixed steel offshore structures are summarized in ISO 19900. Operational requirements include

a) service requirements, i.e. function of the platform,

b) manning,

c) conductors and risers,

d) equipment and material layouts, including rig access to conductor slots,

e) personnel and material transfer,

f) motions and vibrations,

g) any special requirements, and

h) location and orientation.

In addition, requirements for decommissioning and removal shall be considered.

6.3.2 Water depth

The water depth, surge height and tide shall be determined as accurately as possible, so that elevations can be established for boat landings, fenders, decks, and corrosion protection; see ISO 19901-1 for information on these parameters. The potential subsidence of the sea floor and settlement of the structure shall be considered when determining the design water depth.

6.3.3 Structural configuration

6.3.3.1 General

Requirements for the structural configuration are given in ISO 19900; some further requirements are given in 6.3.3.2 and 6.3.3.3.

6.3.3.2 Deck elevation

In general, the elevation of the lowest deck, and any facilities or equipment hanging below the lowest deck, shall provide sufficient clearance to allow the passage of wave crests of abnormal values. See A.6.3.3.2.

Whenever this requirement is not satisfied, the structure shall be designed to resist any wave and current actions on the deck(s), including the actions on facilities or equipment supported by the deck(s).

6.3.3.3 Equipment and material layouts

Layouts and centres of gravity associated with permanent and variable actions, as defined in 9.2, are required in the development of the design. Heavy concentrated actions on the structure shall be located such that proper framing to support these actions can be planned. Consideration shall be given to maintenance requirements and future operations.

If portable equipment or materials are to be placed on a lower deck, then adequately sized hatches shall be provided and conveniently located in the upper decks.

6.3.4 Access and auxiliary systems

The number and location of stairways and access boat landings on the structure shall, as a minimum, be governed by safety considerations. Consideration shall be given to providing access to the water level for sea escape and to facilitate self-rescue in man-overboard situations. Furthermore, operating requirements as well as installation, maintenance and inspection requirements shall be considered in determining the number and location of access platforms, walkways and stairways.

6.4 Safety considerations

6.4.1 General

The safety of life, environment, and property depends upon the ability of the structure to withstand the actions for which it was designed and to survive the environmental conditions which can occur. Good practice dictates the use of certain structural additions, equipment, and operating procedures such that injuries to personnel and the risks of accidental events are minimized.

6.4.2 Accidental events

Personnel safety and possible damage to or loss of the structure require that attention be given to accidental events as summarized in ISO 19900. With regard to methods of protecting against fires and explosions, the selection of the system depends upon the function of the platform. Procedures should conform to any applicable national or regional requirements, as well as being in accordance with ISO 13702.

6.5 Environmental considerations

6.5.1 General

Information on the consideration of environmental conditions and their use is provided in ISO 19900. Further requirements and information are contained in ISO 19901-1, as well as in this International Standard.

6.5.2 Selecting design metocean parameters and action factors

The reliability of a structure depends on the combination of design actions and design resistances. The design actions are the product of selected representative actions and associated partial action factors. Representative environmental actions are governed by the selection of appropriate metocean (meteorological and oceanographic) parameters for design. As a guide, the recurrence interval for metocean design parameters should be several times the design service life of the structure.

Design in accordance with this International Standard shall be based on a return period of 100 years. Where the data are available and sufficient, the 100 year return period may apply to responses (action effects) of the structure instead of to metocean design parameters. Further requirements and guidance are provided in Clause 9 and in ISO 19901-1.

In some cases, lower return periods may be considered, provided that the partial action factors used and the deck elevation selected give equivalent structural reliability for the exposure level of the platform.

Aspects to be considered in selecting partial action factors are

a) the intended use of the structure,

b) the structure design service life,

c) the time and duration of construction, installation, and actions due to environmental conditions during operations,

d) the exposure level,

e) the requirements of regulators,

f) the uncertainty in actions associated with specific metocean parameters and with operating conditions, and the ability to predict the structure’s resistance to the actions,

g) the probability of occurrence and magnitude of extreme and abnormal metocean actions, taking into account the joint frequency of occurrence of extreme winds, waves and currents (both in magnitude and direction),

h) the probability of occurrence and magnitude of actions due to extreme and abnormal earthquakes, see ISO 19901-2, and

i) the probability of occurrence and magnitude of extreme and abnormal actions due to sea ice, see ISO 19906 [1].

In addition to specifying extreme and abnormal metocean parameters, the owner shall specify the limiting environmental conditions for performing any particular operation inducing specific actions, or for any particular situation of the platform.

6.6 Exposure levels

6.6.1 General

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

The life-safety category addresses personnel on the platform and the likelihood of successful evacuation before a design environmental event occurs.

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

6.6.2 Life-safety categories

The category for life-safety shall be selected from the following options with specific requirements.

a) S1 Manned non-evacuated

The manned non-evacuated category refers to a platform that is continuously (or nearly continuously) occupied by persons accommodated and living thereon, and from which personnel evacuation prior to the design environmental event is either not intended or impractical.

A platform shall be categorized as S1 manned-non-evacuated unless the particular requirements for S2 or S3 apply throughout the design service life of the platform.

b) S2 Manned evacuated

The manned evacuated category refers to a platform that is normally manned except during a forecast design environmental event. For categorization purposes, a platform shall not be categorized as a manned evacuated platform unless

1) reliable forecast of a design environmental event is technically and operationally feasible, and the weather between any such forecast and the occurrence of the design environmental event is not likely to inhibit an evacuation,

2) prior to a design environmental event, evacuation is planned,

3) sufficient time and resources exist to safely evacuate all personnel from the platform and all other platforms likely to require evacuation for the same storm.

e) S3 Unmanned

The unmanned category refers to a platform that is only manned for occasional inspection, maintenance and modification visits. For categorization purposes, a platform shall not be categorized as unmanned unless

1) visits to the platform are undertaken for specific planned inspection, maintenance or modification operations on the platform itself,

2) visits are not expected to last more than 24 h during seasons when severe weather can be expected to occur,

3) the evacuation criteria 1) to 3) for S2 manned evacuated platforms are also met.

A platform in this category may also be described as “not normally manned”.

It is recognized that life-safety category definitions include a degree of judgment. The owner of the structure shall determine the applicable category prior to the design of a new structure or the assessment of an existing structure and shall obtain the agreement of the regulator where applicable.

6.6.3 Consequence categories

Factors that should be considered in determining the consequence category include

— life-safety of personnel on or near to the platform brought in to react to any consequence of failure, but not personnel that are part of the normal complement of the platform,

— damage to the environment, and

— anticipated losses to the owner, to other installation owners, to industry and/or to other third parties as well as to society in general.

The consequence category shall be selected from the following options with specific requirements.

a) C1 High consequence category

The high consequence category refers to platforms with high production rates or large processing capability and/or those platforms that have the potential for well flow of either oil or sour gas in the event of platform failure. In addition, it includes platforms where the shut-in of the oil or sour gas production is not planned, or not practical, prior to the occurrence of the design event (such as areas with high seismic activity). Platforms that support trunk oil transport lines and/or storage facilities for intermittent oil shipment are also considered to be in the high consequence category.

A platform shall be categorized as C1, high consequence, unless the particular requirements for C2 or C3 apply throughout the design service life of the platform.

b) C2 Medium consequence category

The medium consequence category refers to platforms where production can be shut-in during the design event. For categorization purposes, a platform shall not be categorized as medium consequence unless

1) all wells that can flow on their own in the event of platform failure contain fully-functional subsurface safety valves, manufactured and tested in accordance with applicable specifications,

2) oil storage is limited to process inventory and “surge” tanks for pipeline transfer,

3) pipelines are limited in their ability to release hydrocarbons, either by virtue of inventory and pressure regime, or by check valves or seabed safety valves.

c) C3 Low consequence category

The low consequence category refers to minimal platforms where production can be shut-in during the design event. These platforms may support production departing from the platform and low volume in-field pipelines. For categorization purposes, a platform shall not be categorized as low consequence unless

1) all wells that can flow on their own in the event of platform failure contain fully-functional subsurface safety valves, manufactured and tested in accordance with applicable specifications,

2) oil storage is limited to process inventory,

3) pipelines are limited in their ability to release hydrocarbons, either by virtue of inventory and pressure regime, or by check valves or seabed safety valves.

It is recognized that consequence category definitions include a degree of judgment. The owner of the structure shall determine the applicable category prior to the design of a new structure or the assessment of an existing structure and shall obtain the agreement of the regulator where applicable.

6.6.4 Determination of exposure level

The three categories for both life-safety and consequence can, in principle, be combined into nine exposure levels. However, the level to be used for structure categorization is the more restrictive level for either life-safety or consequence.

This results in three exposure levels as illustrated in Table 6.6-1:

Table 6.6-1 — Determination of exposure level

Life-safety category	Consequence category		
	C1 High consequence	C2 Medium consequence	C3 Low consequence
S1 Manned non-evacuated	L1	L1	L1
S2 Manned evacuated	L1	L2	L2
S3 Unmanned	L1	L2	L3

The exposure level applicable to a structure shall be determined by the owner prior to the design of a new structure or the assessment of an existing structure, and be agreed upon by the regulator where applicable.

Platform categorization may be revised over the design service life of the structure as a result of changes in factors affecting life-safety or consequence category.

6.7 Assessment of existing structures

Any assessment of existing structures to confirm that they comply with this International Standard or are fit-for-purpose shall be performed in accordance with the assessment requirements of ISO 19900 and with Clause 24.

6.8 Structure reuse

Structures may be removed and relocated for use at a new location. When this is considered, the structure shall be assessed in accordance with the assessment requirements of ISO 19900 and Clause 25, for the use (including exposure level) and conditions that are applicable at the new location. Any repairs or modifications that are necessary shall be in accordance with this International Standard.

7 General design requirements

7.1 General

The general principles on which structural design requirements are based are documented in ISO 19900. ISO 19900 requires that structural design be performed with reference to a specified set of limit states. For each limit, state design situations shall be determined and an appropriate calculation model shall be established. ISO 19900 describes

— the variables that occur in a calculation model, comprising geometrical parameters, actions and properties of materials and soils,

— the analyses to be performed, and

— the design format using partial action and partial resistance factors.

This clause outlines the overall requirements for

a) incorporating limit states (see 7.2),

b) determining design situations (see 7.3),

c) structural modelling and analysis (see 7.4), and

d) design of the structure (see 7.5 to 7.11).

These requirements are a minimum.

7.2 Incorporating limit states

ISO 19900 divides the limit states into four categories, as below.

a) Ultimate limit states (ULS)

The design actions to be used in the various ULS are specified in Clauses 8, 9 and 11. The design resistances and the application of the ULS are specified in Clauses 13, 14, 15 and 17.

f) Serviceability limit state (SLS)

The design shall comply with the serviceability requirements of caissons, conductors, risers and topsides interfaces, as defined by the owner.

g) Fatigue limit states (FLS)

The various FLS are addressed in Clause 16, covering methods, actions and resistances. Additionally, FLS requirements for grouted connections, mechanical connections and clamps are given in Clause 15.

h) Accidental limit states (ALS)

ALS are addressed in Clause 10, in respect of actions and modifications to both partial action factors and partial resistance factors.

7.3 Determining design situations

Determination of situations for which structures are to be designed is the responsibility of the owner in accordance with the requirements of a regulator, where one exists. Aspects to be considered in determining design situations include

a) service requirements for the intended function of the structure,

b) expected service life for each function,

c) method and duration of construction activities,

d) expected method of removal of the structure and, where applicable, any intended relocation,

e) hazards (accidental and abnormal events) to which the structure can be exposed during its service life,

f) potential consequences of partial or complete structural failure, and

g) the nature and severity of environmental conditions (meteorological, oceanographic and active geological processes) to be expected during its construction and service life.

Usually, three sets of environmental conditions should be considered: one associated with operating situations (normal conditions), one associated with extreme conditions and one associated with abnormal conditions.

7.4 Structural modelling and analysis

Internal forces in components shall be determined by an analysis method that is suitable for statically indeterminate structures and that accounts for bending as well as for axial action effects. In general, a linear elastic model of the structure is sufficient; however, the non-linear behaviour of structure-foundation interaction shall be addressed. For piled foundations the non-linear behaviour of the axial and lateral pile-soil support shall be explicitly modelled to ensure load-deflection compatibility between the structure and pile-soil system. For a pile analysis, the effects of geometrical and material non-linearities shall also be considered within the structure-pile-soil system. In no case shall additional or adjusted partial action factors be used as a substitute for a rational analysis to determine internal forces. Other aspects of structural modelling and analysis are discussed in Clauses 8 to 17 and 24.

7.5 Design for pre-service and removal situations

The structure shall be designed to resist actions occurring during fabrication, loadout, transportation, installation and removal, as specified in Clause 8. For structures that are intended to be relocated to new sites, actions resulting from removal, on-loading, transportation, upgrading, and re-installation shall also be considered in the design, in addition to the above construction actions.

Methods for fabrication, assembly, loadout, transportation, installation and removal of the structure shall be considered during design in order to identify potential difficulties and limitations for available equipment. Design situations for partially fabricated components of the structure and for the assembled structure as a whole shall be established. Fabrication and assembly tolerances assumed in design shall be compatible with those specified in the construction specifications and achievable during construction.

7.6 Design for the in-place situation

The structure shall be designed to resist permanent actions, variable actions, environmental actions, repetitive actions and accidental actions occurring during its service life, as well as rational combinations thereof, to obtain the most onerous conditions for all structural components. The representative values of these actions are given in Clauses 9 to 11. Clarification on the selection of representative values and return periods is provided in A.7.6.

7.7 Determination of resistances

7.7.1 General

The resistances of components in a fixed steel offshore structure shall, wherever possible, be derived from the formulae given in Clauses 13 to 15. These formulae provide representative resistances to which the partial resistance factors are applied.

Where the resistance of a component is not adequately described in Clauses 13 to 15, the resistances shall be determined from a rational study of the behaviour of the component. Both physical testing and computer simulation may be used to derive representative resistances, but in both cases the results shall be compared to any other available test data.

7.7.2 Physical testing to derive resistances

Physical tests may be used to derive resistances for specific components. Care shall be taken to account for the differences between the test arrangements and the arrangement of the component within the structure. Guidance on these differences is given in A.7.7.2.

Comparisons shall be made with other available test data to give an indication of the variability of the component behaviour.

7.7.3 Resistances derived from computer simulations validated by physical testing

Computer simulations can provide an indication of resistance and can be particularly valuable for investigating the effects of changes of various parameters. Computer simulations shall be validated against physical test data wherever possible. If test data are not available, the requirements of 7.7.5 apply.

7.7.4 Resistances derived from computer simulations validated against design formulae

As an alternative to validation against physical test data, computer simulations may be validated against the formulae given in Clauses 13 to 15. It should be noted that Clauses 13 to 15 give representative resistances, and therefore take account of structural behaviour variability.

7.7.5 Resistances derived from unvalidated computer simulations

Unvalidated computer simulations may only be used when no physical test data or design formulae are available. The similarity of available data and the component being simulated is a matter of judgment, and the analyst shall take account of statistical variability of material and component behaviour, and limitations and errors that can be introduced in computer simulations.

7.8 Strength and stability checks

7.8.1 Action and resistance factors

A partial action factor shall be applied to each of the external actions in the combinations given in Clauses 8 to 11. The combination of factored representative actions results in design values of internal forces. These internal forces are designated by the symbol S in Clauses 8 to 11.

A partial resistance factor is applied to the strength of each member, joint and foundation component to determine its design resistance.

7.8.2 Strength and stability equations

Each component shall be proportioned to resist the internal forces, S. The appropriate strength and stability equations are given in Clauses 13, 14, 15 and 17. These equations comprise

a) the formulae for the representative strength of the component,

b) the partial resistance factors, and

c) the internal force, S, from the design actions specified in Clauses 8 to 11.

7.8.3 Unfactored actions

Where actions are time-varying and non-synchronous, the most onerous values of action effects cannot generally be determined by simply adding factored individual actions. In some construction and installation situations, the action effects are best computed from unfactored actions and by investigating the sensitivity of the action effects with realistic variations of the governing parameters. The design values of the internal forces, S, may then be obtained by factoring the action effects by the partial action factors given in Clause 8.

7.9 Robustness

A structure shall incorporate robustness through consideration of the effects of all hazards and their probabilities of occurrence, to ensure that consequent damage is not disproportionate to the cause. Damage from an event with a reasonable likelihood of occurrence shall not lead to complete loss of integrity of the structure. In such cases, the structural integrity in the damaged state shall be sufficient to allow a process system close down and a safe evacuation, see Clause 10.

Robustness is achieved by either

a) designing the structure in such a way that any single load bearing component exposed to hazard can become incapable of carrying its normal design actions without causing collapse of the structure or any significant part of it, or

b) ensuring (by design or by protective measures) that no critical component exposed to hazard can be made ineffective, or

c) a combination of a) and b), above.

7.10 Reserve strength

7.10.1 New structures

All new structures complying with this International Standard are intended to have a similar margin of system reserve strength for the same exposure level (L-level). The system reserve strength gives the structure a margin to withstand environmental action effects that exceed the design action effects and is evaluated as the reserve strength ratio (RSR), R_{RS}, expressed in Equation (7.10-1).

$$R_{\mathrm{RS}} = F_{\mathrm{collapse}}/F_{100} \tag{7.10-1}$$

where

F_{collapse} is the unfactored global environmental action which, when co-existing unfactored permanent and variable actions are added, causes collapse of the structure;

F_{100} is the unfactored 100 year global environmental action calculated in accordance with Clause 9.

This intent may be achieved in either of two ways, as described in 9.10. The partial action factors for environmental actions shall be determined based on the considerations given in Clause 9.

When using this International Standard for structures that provide less inherent system reserve strength (smaller RSR) for the environmental actions than a typical space frame type structure, the partial action factor for environmental actions shall be increased to yield the same RSR as for a typical space frame type structure.

7.10.2 Existing structures

During its service life, the reserve strength of a structure can vary (increase or decrease) due to a variety of causes. Clause 24 provides requirements for the assessment of existing structures, including the use of system strength approaches.

7.11 Indirect actions

Consideration shall be given to internal forces and stresses induced by indirect actions, including imposed or constrained deformations, temperature changes, relaxation, and prestressing.

7.12 Structural reliability analysis

Structural reliability analysis (SRA) methods are used to assess the effects of uncertainties in the actions, resistances and modelling of (parts of) a structure and its performance. Uncertainties exist due to the natural variation, the physical uncertainty or the randomness in the basic variables (aleatoric or Type I uncertainties), and due to those factors that are a function of a lack of complete understanding or knowledge (epistemic or Type II uncertainties).

SRA is not normally undertaken as part of a new design in accordance with this International Standard, but may be used during the initial design process to provide comparative data. SRA may be used in the (re-)calibration of partial action and resistance factors for special or unusual circumstances, in decision analyses to support inspection and monitoring programmes, and in some cases in the structural assessment of existing structures. The design procedures in this International Standard shall be used for new designs.

The objective of SRA is to determine the probability of an event (usually a failure event) occurring during a specified reference period. The failure probability during the structure's design service life (or for some other specified reference period) can be of use in economic risk models; annual failure probabilities are generally evaluated when considering life-safety issues.

Where SRA methods are used, it is generally necessary to compare evaluated failure probabilities with appropriate target values. Target reliabilities should allow for the consequences and nature of failure, and where possible, should be calibrated against well established cases that are known to have adequate safety.

8 Actions for pre-service and removal situations

8.1 General

8.1.1 Coverage

This clause presents requirements for pre-service and removal situations. The requirements apply to both the structure and to any temporary components such as buoyancy aids attached to the structure for pre-service or removal situations. The primary objective is to ensure that a structure begins its service life at an offshore location with its required strength and structural integrity intact. The clause covers the following topics, following the requirements of ISO 19900:

a) the type of actions to be included;

b) procedures for determining representative values for the actions;

c) the partial action factors to be applied;

d) methods for determining the internal forces due to the effect of factored actions or combinations of factored actions.

8.1.2 Situations

Design situations to be considered during pre-service and removal situations of the structure are transient situations, in accordance with ISO 19900, and are listed below.

a) Pre-service situations comprise

   1) fabrication (including assembly during fabrication),

   2) loadout (including float-out),

   3) transportation, and

   4) installation (including launching and uprighting).

i) Removal situations comprise

1) removal prior to relocation to a new location for a new period of service, and

2) removal at the end of service life.

Further requirements concerning pre-service situations are given in Clauses 19 to 21.

Actions on (parts of) the structure and internal forces in components of the structure shall be calculated for each phase of pre-service or removal situations in accordance with the principles given in 7.3 to 7.5.

The requirements of 8.2 and 8.3 apply to all pre-service and removal situations. Specific requirements for each phase of pre-service situations are subsequently given in 8.4 to 8.7. Requirements for removal situations are given in 8.8.

8.2 General requirements

8.2.1 Design situations

Design situations during pre-service and removal situations are generally governed by permanent and variable actions. These actions are predominantly static actions, in particular due to weights of structural components, appurtenances and equipment. When dynamic effects can be important, these shall be considered. When appropriate, environmental and accidental actions shall also be given due attention, following the requirements of Clauses 9 and 10.

8.2.2 Weight control

The weight and centre of gravity of each structure or part of a structure shall be evaluated during design and during all pre-service and removal situations. This applies in particular prior to any lifting being performed. Where applicable, allowances for weight variations based on weight reports, the status of design and/or fabrication, known outstanding items and/or operational modifications to the structures should be applied.

During pre-service and removal situations the behaviour of the structure and the magnitude and distribution of internal forces in the structure can be very sensitive to the position of the centre of gravity. In such cases, due allowances shall be made for uncertainties and potential changes in the position of the centre of gravity.

NOTE Guidance on weight control is given in ISO 19901-5[5].

8.2.3 Dynamic effects

If operations during pre-service and removal situations involve significant motion of large masses, the dynamic actions involved shall be considered.

The action effects shall be adjusted to arrive at adequate representative internal forces for the design of the components affected. These representative internal forces can be derived from an appropriate analysis to determine the static and dynamic effects due to both the external actions and the motion, or they can be based on relevant previous experience.

For dynamic effects during lifting, see the requirements given in 8.3.2.

8.2.4 Internal forces

8.2.4.1 Internal forces due to factored actions

Each member, joint and other relevant component shall be checked for strength using the internal force (action effect) (S) resulting from the design action (F_d).

The design action, F_d, is due to the following:

— factored permanent actions (G);

— factored variable actions (Q);

— factored quasi-static transient actions (T).

Equation (8.2-1) defines F_d for situations not involving lift (additional factors are applied for lift conditions, see 8.3.6):

$$F_d = \gamma_{f,GT}\, G_T + \gamma_{f,QT}\, Q_T + \gamma_{f,T}\, T \qquad (8.2\text{-}1)$$

where

G_T is the action imposed either by the weight of the structure in air, or by the submerged weight of the structure in water, during the transient situation being considered, including any permanent equipment or other objects and any piles or conductors installed on the structure, as well as any ballast installed in or carried by the structure;

Q_T is the action imposed by the weight of the temporary equipment or other objects, including any rigging installed or carried by the structure, during the transient situation being considered;

T represents the actions from the transient situation being considered, including:

a) when appropriate, environmental actions,

b) when appropriate, a suitable representation of dynamic effects (see A.8.1 and 8.2.4.2),

c) for lifting, the effects of fabrication tolerances and variances in sling length as detailed in 8.3.3 and for a dual lift as detailed in 8.3.4,

d) for loadout, allowances for misalignment as detailed in 8.5,

e) for transportation, any hydrostatic and hydrodynamic actions on the structure, including any inertial actions resulting from accelerations of the structure (see 8.6), and

f) for installation, the lifting actions and hydrostatic pressure actions on the structure (see 8.7);

$\gamma_{f,GT}$, $\gamma_{f,QT}$ and $\gamma_{f,T}$ are the partial action factors.

The three design situations in Table 8.2-1 shall all be considered.

Table 8.2-1 — Partial action factors for calculating internal forces

Situation	Partial action factor		
	$\gamma_{f,GT}$	$\gamma_{f,QT}$	$\gamma_{f,T}$
1	1,3	1,3	1,0
2	1,1	1,1	1,35
3	0,9	0,9	1,35

NOTE Situation 1 governs for components in which permanent and variable action effects are dominant. Situation 2 governs for components in which transient action effects are dominant and in which the permanent and variable actions increase the magnitudes of the internal forces. Situation 3 governs for components in which transient action effects are dominant but in which the permanent and variable actions decrease the magnitudes of the internal forces.

Additional requirements and factors for lifting situations are given in 8.3.

8.2.4.2 Internal forces due to unfactored actions

In some situations it is not possible or appropriate to apply action factors to each of the individual actions, see A.8.1 and 7.8.3. Examples of such situations are transportation, launching or lifting in which substantial dynamic or non-linear effects can occur. In such cases each member, joint and other relevant component shall be checked for strength using the internal force (action effect) (S) determined from the following equations:

$$F_{\mathrm{un}} = G_{\mathrm{T}} + Q_{\mathrm{T}} + T$$

$$S = \gamma_{\mathrm{f,Sun}}\, S_{\mathrm{un}} \qquad (8.2\text{-}2)$$

where

F_{un} is the total action due to the unfactored actions G_{T}, Q_{T} and T defined in 8.2.4.1;

S_{un} is the internal force resulting from F_{un};

$\gamma_{\mathrm{f,Sun}}$ is the partial factor to be applied to S_{un}, usually 1,3.

Additional requirements and factors for lifting situations are given in 8.3.

If both this method and the one according to 8.2.4.1 can be applied, that giving the more onerous result shall be used.

8.3 Actions associated with lifting

8.3.1 General

Lifting imposes static and dynamic actions on the structure during various stages of pre-service and removal situations. The magnitude of these actions shall be based on the weight of the (part of the) structure to be lifted and shall be determined considering static and dynamic equilibrium during the lift. The internal forces in the structure and in the lifting slings and rigging equipment (e.g. spreader bars) shall be derived from a rational analysis of the lifting arrangement. General guidance on lifting is given in A.8.3.1.

Dynamic amplification (8.3.2), as well as the effects of fabrication and sling length tolerances (8.3.3), and weight and centre of gravity location inaccuracies (8.2.2), shall be included in such rational analysis.

Additional factors and considerations apply to members adjacent to lifting attachments (8.3.5), and to the lifting attachments themselves (see 8.3.6 and 8.3.7).

Alternative lift criteria can be appropriate, for example when the characteristics of, and interactions between, the transportation barge, structure and crane are fully understood. Alternative criteria may be used, subject to demonstration of an acceptable margin of safety and to the agreement of the owners of the structure, the contractors and any appropriate regulator or warranty surveyor.

8.3.2 Dynamic effects

A dynamic amplification factor (DAF), k_{DAF}, accounting for dynamic effects of the crane taking up the load and for movements of the crane or of the lifted structure, shall be derived from the following.

a) For offshore lifts in air:

1) $k_{\mathrm{DAF}} = 1{,}10$ for heavy lift by semi-submersible crane vessel;

2) $k_{\mathrm{DAF}} = 1{,}30$ in other cases.

j) For lifts in air, onshore or in sheltered waters:

$$k_{DAF} = 1{,}10$$

k) For lifts partially or wholly in water, k_{DAF} shall be specially investigated taking into account factors including

— the lift arrangement,

— the orientation of the lifted structure,

— the ratio of the allowable hook load to the lifted weight (including the effect of any hydrodynamic added mass),

— the drag loads on the lifted structure, and

— the motions of the boom tip in the environmental conditions in which the lift is to be made.

Special investigations may be performed to substantiate a lower DAF value for case a) 2) above; however, k_{DAF} shall not be less than 1,10.

See also ISO 19901-6[3].

8.3.3 Effect of tolerances

Fabrication misalignments in the structure and sling length variances both affect the distribution of forces in the lifting system. The requirements and partial action factors specified in 8.2.4, 8.3.1 and 8.3.2 apply to situations where fabrication misalignments are consistent with the tolerances specified in Annex G and where the variance on the length of slings does not exceed the greater of 0,25 % of the nominal sling length or 40 mm.

The effect of sling variances in statically indeterminate lift systems (e.g. four slings from four lift points to one hook) shall be taken into account. Sling forces or sling lengths shall be varied in a stiffness analysis of the structure and lifting system so as to determine the redistribution of forces in both slings and structural members. The tolerances in sling length shall be applied to the slings to maximize the sling forces, see A.8.3.3.

The resulting sling force should be increased by a factor of not less than 1,25 (1,15 for floating spreader beams) to determine the required safe working load of the sling in 8.3.8; the local effects on the structure shall be checked using the local factor given in 8.3.5.

8.3.4 Dual lift

For a dual lift from a floating vessel (i.e. a lift operation with two cranes on a single crane vessel), an additional rigging factor should be considered in lieu of assessment of the effects of tilt and yaw on lift forces. Unless special and reliable investigations are performed to substantiate lower values, a minimum value of such a rigging factor of $\gamma_{f,dl} = 1{,}10$ should be applied. For single crane lifts, $\gamma_{f,dl} = 1{,}00$.

Where a dual lift is used for uprighting a structure, the most onerous orientations of the structure shall be considered and the design of lifting attachments shall allow for any necessary rotation of the lifting arrangements.

8.3.5 Local factor

Due to the local inaccuracies which often occur in the analysis of internal force distributions around the lifting points, a further local factor ($\gamma_{f,lf}$) shall be applied in order to account for the importance of the structural components involved. This local factor includes the effect of tolerances (see 8.3.3), but shall be applied in addition to the DAF (k_{DAF}) in accordance with 8.3.2 and the rigging factor ($\gamma_{f,dl}$) in accordance with 8.3.4.

The local factor, both for statically determinate and statically indeterminate lifts (where the effect of tolerances on the force distribution is taken into account in the analysis in accordance with 8.3.3), is given below.

a) For lifting attachments (padeyes, trunnions, padears), spreader beams, and internal members (including both end connections) framing into the joint where the lifting attachment is attached and transmitting lift forces:

$\gamma_{f,lf}$ = 1,25 (for a lift in open waters);

$\gamma_{f,lf}$ = 1,15 (for a lift onshore or in sheltered waters).

l) For other structural members:

$\gamma_{f,lf}$ = 1,00

8.3.6 Member and joint strengths

The DAF (k_{DAF}), the rigging factor ($\gamma_{f,dl}$) and the local factor ($\gamma_{f,lf}$) shall be applied to either the factored actions to determine the design action (F_d) as in Equation (8.2-1), giving Equation (8.3-1), or to unfactored actions to give the internal forces (S) as in Equation (8.2-2), giving Equation (8.3-2):

$$F_d = k_{DAF}\, \gamma_{f,dl}\, \gamma_{f,lf}\, (\gamma_{f,GT}\, G_T + \gamma_{f,QT}\, Q_T + \gamma_{f,T}\, T) \tag{8.3-1}$$

$$S = k_{DAF}\, \gamma_{f,dl}\, \gamma_{f,lf}\, \gamma_{f,Sun}\, S_{un} \tag{8.3-2}$$

where

$\gamma_{f,GT}$, G_T, $\gamma_{f,QT}$, Q_T, $\gamma_{f,T}$, T, $\gamma_{f,Sun}$, S_{un} are as defined in 8.2.4;

k_{DAF} is as specified in 8.3.2;

$\gamma_{f,dl}$ is as specified in 8.3.4;

$\gamma_{f,lf}$ is as specified in 8.3.5.

8.3.7 Lifting attachments

Lifting attachments can be of various forms, including the following.

a) **Padeyes**, where a shackle pin passes through a hole in a padeye plate attached to or built into the structure, the sling being connected to the shackle. The padeye plate is normally reinforced to increase strength and avoid undue slack between the shackle and the padeye, thereby reducing eccentricity between the padeye plate and the sling.

b) **Trunnions**, where the sling, or an eye of a sling, passes round a short tubular which transfers the forces into the structure, and which allows rotation of the sling around the axis of the trunnion. Trunnions can be arranged to carry either end of a sling on either side of a central plate.

c) **Padears**, which are similar to trunnions, but in which rotation of the sling is not intended.

Padeye plates shall be oriented in such a direction that the possibility for out-of-plane loading of the padeye plate and shackle is minimized.

To account for any side loading on lifting attachments, lifting attachments and the connections to the supporting structural members shall be designed for a lateral force of 5 % of the sling force, in addition to the calculated horizontal and vertical components of the sling force (including DAF, rigging factor, local factor and partial action factors) for the equilibrium lifting condition. This lateral force acts simultaneously with the static sling force and shall be applied perpendicular to the lifting attachment at the centre of the pinhole or tubular. Where a spreader bar is directly connected to the padeyes, a lateral force of 8 % shall be used.

Where two slings are connected to one padeye, or where a sling is doubled over a trunnion, the padeye or trunnion should be designed for a 45:55 % split of the lift point force between the two slings.

8.3.8 Slings, shackles and fittings

For normal offshore conditions, slings should have a total resistance factor of 4,0 on the manufacturer's rated minimum breaking strength of the cable compared to the calculated sling force. The calculated sling force is the maximum force on each sling, as calculated in 8.3.1 to 8.3.4, by taking into account all actions, the equilibrium position of the lift, any dynamic amplification, the effect of tolerances and dual lift considerations. This total resistance factor includes all necessary partial resistance factors. The total resistance factor should be increased when unusually severe conditions are anticipated. Conversely, the total resistance factor may be reduced to a minimum of 3,0 for carefully controlled conditions.

Where two slings are connected to one padeye, or where a sling is doubled over a trunnion, the slings should be assumed to carry the lift point force in a 45:55 % split of the lift point force between the two slings.

Shackles and fittings should be such that the manufacturer's rated working load is greater than or equal to the calculated sling force, provided the manufacturer's specifications include a minimum resistance factor of 3,0 on the minimum breaking strength.

8.4 Actions associated with fabrication

In addition to the associated permanent and variable actions, the effects of wind-induced vortex shedding vibrations on long slender members during fabrication should be considered.

8.5 Actions associated with loadout

8.5.1 Direct lift

Actions on a structure that is lifted onto the transportation barge shall be evaluated in accordance with 8.3. If the lifting arrangement is the same as that used to offload the structure from the transportation barge at sea, it will normally suffice to check the latter load case(s) only. See 8.7.1.

8.5.2 Horizontal movement onto barge

Structures skidded onto transportation barges are subject to actions resulting from horizontal and/or vertical movement of the barge due to tidal fluctuations, nearby marine traffic, environmental conditions and/or changes in draught, as well as to actions derived from the location, slope, and/or settlement of supports onshore and the flexibility of the barge, at all stages of the skidding operation.

The effect of actions on the structure during loadout shall be investigated, taking account of the tolerances on the levels of support on land and on the barge. The effect of pulling/pushing, friction forces and twisting of the structure due to differential horizontal jacking or winching during loadout shall be considered.

In cases of loadout with trailers, the effect of distributed actions imposed by hydraulic trailers and the effect of dynamic actions due to the braking of the trailers should be considered. This can require consideration of the methods of controlling the elevation and alignment of trailers.

Since movement is normally slow, impact does not need to be considered.

8.5.3 Self-floating structures

Self-floating structures are generally not transported on a barge but are towed while floating to the offshore location.

Self-floating structures skidded directly into the water at the fabrication yard shall be analysed to determine the actions on the structures as they move down the slipways and into the floating position. Consideration should be given to local environmental conditions and dynamically induced forces.

This subclause does not apply to self-floating structures built in a dry dock and floated by flooding the dock.

8.6 Actions associated with transportation

8.6.1 General

Actions on a structure or structural part during transportation shall be considered in the design of the structure, whether it is transported on barges or is self-floating. Such actions result from the way in which the structure is supported, either by barge or by buoyancy, and from the response of the towing arrangement (structure, tug boats, tow lines and, where applicable, barge and/or additional buoyancy) to environmental conditions encountered en route to the site.

8.6.2 Environmental conditions

For the selection of environmental conditions to be used in determining the motions of the towing arrangement and the resulting actions on the structure, the sea-fastenings and the barge system, the following should be considered:

a) previous experience along the tow route;

b) exposure time and reliability of predicted weather windows;

c) accessibility of safe havens;

d) seasonal weather systems;

e) appropriateness of the recurrence interval used in determining design wind, wave and current conditions in relation to the characteristics of the tow.

8.6.3 Determination of actions

The motions of the towing arrangement due to the environmental conditions determined from 8.6.2 shall be analysed using suitable methods. The motions should be based on the results of model basin tests or appropriate analytical methods. Analytical methods shall use spectral analysis techniques to account for the distribution of wave energy over frequency; when appropriate, the distribution of wave energy over wave directions shall also be considered. Sea states with peak periods corresponding to the natural periods of the loaded barge, as well as peak periods corresponding to wave lengths which are critical in relation to the dimensions of the barge, shall be considered, as these can represent governing conditions. In combining responses, the analysis shall correctly account for relative phasing between motion components (heave, pitch, roll, etc.).

Head, beam and quartering winds and seas should be considered to determine maximum responses due to the environmental actions on the overall system. In cases of large barge-transported structures, the stiffnesses of both the structure and the barge shall be included in the structural analysis.

Actions on the structure, sea-fastenings and the barge system shall be derived from appropriate combinations of permanent, environmental and inertial actions resulting from the state of motion, taking due account of the position, orientation and possible differential deformations of parts of the overall system.

Where circumstances and experience make such simplifications reasonable, tows may be analysed for strength, based on a combination of permanent and inertial actions resulting from the tow's rigid body motions and determined using appropriately selected single period(s) and amplitude(s), and combining

a) roll with heave, and

b) pitch with heave.

8.6.4 Other considerations

Large structures overhanging the barge are usually subjected to partial submersion during tow. Submerged members should be investigated for actions due to slamming and buoyancy. These actions should be included in the local and, where relevant, overall design of the structure and the sea-fastenings. Large buoyant overhanging members can also affect motions and this effect should also be considered. The effects of vortex induced vibrations (VIV) due to wind on long slender members also should be investigated.

The effects of repetitive actions during tow can become significant to the fatigue life of certain members, joints or other components, especially for long transoceanic tows. These effects should be investigated using methods and acceptance criteria agreed upon by the owner and the design or transportation contractor.

8.7 Actions associated with installation

8.7.1 Lifted structures

Actions on a structure that is lifted off the transportation barge at sea shall be evaluated in accordance with 8.3.

8.7.2 Launched structures

A structure shall not be launched from a barge if the significant wave height exceeds 2,0 m or if it is expected to exceed 2,0 m before sufficient on-bottom stability is achieved (see 8.7.6).

Barge-launched structures shall be analysed to determine the actions on the structure throughout the launch. Consideration shall be given to hydrostatic pressure (8.7.4), wind and current actions, and the development of dynamically induced actions resulting from the launch.

Horizontal actions required to initiate movement of the structure should also be evaluated. Expected actions on both the structure and the barge during launching should be considered.

8.7.3 Crane assisted uprighting of structures

Where lifting equipment is used to assist in the uprighting of a structure as part of the installation, the structure should be analysed for the permanent and variable actions together with the dynamically induced actions during uprighting. The requirements of 8.3 apply to this situation.

8.7.4 Submergence pressures

Submerged non-flooded, or partially-flooded, components shall be designed to resist pressure-induced stresses during launching and uprighting, in addition to other internal forces acting on the components at the same time.

8.7.5 Member flooding

Consideration shall be given to the effects of unexpected flooding of members or failure of members to flood when intended, either by allowing for such occurrences or by making adequate provisions to prevent such occurrences. Any one failure of the flooding control shall be considered, including common-mode failures such as failure of a valve at a manifold.

8.7.6 Actions on the foundation during installation

8.7.6.1 General

The environmental conditions considered for installation shall be consistent with the limiting environmental conditions in the installation procedure. A more severe environmental condition should also be checked, corresponding to the last stage of installation, when the structure is upright and placed on the sea floor but not yet permanently anchored.

For structures that are to be piled, the actions to be expected from contact with the sea floor during installation of the structure should be assessed to give adequate assurance that the structure will remain at the planned elevation, location and attitude until piles can be installed.

The design shall ensure that

a) the footings or mudmats have adequate capacity against sliding and bearing failure, and that pin-piles, if any, have adequate strength to avoid being damaged,

b) the footings, mudmats, or other bearing components and structural members supporting these, have adequate strength to avoid being damaged, and

c) the safety margins against overturning of the structure are adequate, with the recommendation that the structure be checked in a piled condition but without the permanent action of the topsides if placement of the topsides does not follow shortly after structure installation.

8.7.6.2 Determination of actions

Both vertical and horizontal actions shall be determined, taking into account changes in configuration and exposure, construction equipment and additional ballast that can be required for stability during storms.

8.8 Actions associated with removal

Differences between installation of a new structure and its eventual removal (e.g. the weight of piles and grout within the pile sleeves or structure legs) shall be considered during design to ensure that safe removal is viable. The owner may specify particular removal requirements to be considered during design.

9 Actions for in-place situations

9.1 General

This clause presents actions and combinations of actions and their effects that are to be considered for the in-place situation. It covers the following topics, following the requirements of ISO 19900:

a) the type of actions to be included;

b) procedures for determining representative values for the actions;

c) the partial action factors to be applied;

d) methods for determining the internal forces due to the effect of factored actions or combinations of factored actions.

Design situations to be considered for the structure in-place can be distinguished between persistent situations and accidental situations, in accordance with ISO 19900. In this clause, only the persistent situations are addressed; accidental situations are addressed in Clause 10.

9.2 Permanent actions (G) and variable actions (Q)

9.2.1 Permanent action 1, G_1

G_1 is the action imposed on the structure by the self weight of the structure with associated equipment and other objects. G_1 includes the following:

a) weight of the structure in air, including, where appropriate, the weight of piles, grout, and solid ballast;

b) weight of equipment and other objects permanently mounted on the structure that do not change with the mode of operation;

c) hydrostatic actions acting on the structure below the waterline, including internal and external pressure, and resulting buoyancy;

d) the weight of water enclosed in the structure, whether permanently installed or temporary ballast; see 9.2.5 for unintentional flooding.

The representative value of G_1 is the value computed from nominal dimensions and mean values of densities.

9.2.2 Permanent action 2, G_2

G_2 is the action imposed on the structure by the 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. G_2 includes the following:

a) weight of drilling and production equipment that can be added to or removed from the structure;

b) weight of living quarters, heliport and other life-support equipment, diving equipment, and utilities equipment, which can be added to or removed from the structure.

The representative value of G_2 is the estimated lift weight of the object plus any field installed appurtenances.

9.2.3 Variable action 1, Q_1

Q_1 is the action imposed on the structure by the weight of consumable supplies and fluids in pipes, tanks and stores, the weight of transportable vessels and containers used for delivering supplies, and the weight of personnel and their personal effects. Where appropriate, the weight of marine fouling and ice shall be included in Q_1.

The weight of scaffolding or other temporary access systems used during operations and maintenance of the platform shall also be included in Q_1. The representative value of Q_1 is computed from the nominal weight of the heaviest material and the largest personnel capacity under the mode of operation considered.

9.2.4 Variable action 2, Q_2

Q_2 is the short duration action imposed on the structure from operations, such as lifting of drill string, lifting by cranes, machine operations, vessel mooring, and helicopters. The additional weight of liquids used for testing of vessels and pipes is also included in Q_2.

The representative value of Q_2 is computed from the rated maximum capacity of the equipment involved and includes dynamic and impact effects.

9.2.5 Unintentional flooding

All members that are intended to be internally dry and that are located under the water line shall be checked for strength for the dry as well as for the flooded state.

All flooded members shall also be checked for strength for the dry as well as the flooded state, unless positive means of ensuring full flooding is provided.

In addition, when computing the aggregate weight of the submerged portion of the structure, 10 % of the volume of unflooded members shall be assumed to be flooded if this creates a more severe load case. The weight is to be included as part of G_1.

9.2.6 Position and range of permanent and variable actions

Variations in weights and locations of movable equipment shall be considered in order to determine the maximum internal force, S (action effect), in each component. Maximum and minimum values of G_2, Q_1 and Q_2 for each mode of operation shall be considered, in order to ensure that the most onerous condition for each structural component is determined.

9.2.7 Carry down factors

Where permanent or variable actions can be moved, or their magnitudes can vary, it is conservative to apply the maximum values over the whole area of their influence. In such cases, a reduction in the resulting actions may be used if operating practices provide adequate safeguards to prevent action effects from exceeding the reduced values.

9.2.8 Representation of actions from topsides

Distributed actions over an area may be used to represent permanent and variable actions on platform decks. Partial action factors for distributed actions shall be based on the proportion of permanent and variable actions represented by the distributed action.

Concentrated actions on the deck, for example from separators, may be included within the distributed actions on the topsides for the design of the structure. If a combined analysis model of the structure and topsides is used, concentrated actions should be represented as such with the appropriate partial action factor based on the proportion of permanent and variable actions represented by the concentrated action.

9.2.9 Weight control

During design for in-place situations:

a) the weight of the structure or part of the structure shall be evaluated using a rational weight-estimating procedure, and a factor for weight growth based on a weight report should be applied;

b) the centre of gravity of the structure or part of the structure shall be evaluated using a rational procedure. Allowances shall be made for uncertainties and potential changes in the centre of gravity position.

NOTE Guidance on weight control is given in ISO 19901-5 [5].

9.3 Extreme environmental action due to wind, waves and current

9.3.1 General

Wind, wave and current conditions for calculating extreme environmental actions shall be selected in accordance with ISO 19901-1.

Environmental actions, notably those caused by waves, vary with time. Structural responses (action effects) will therefore also vary with time and will be subject to some degree of dynamic amplification. However, in many cases, the dynamic amplification of the global structural responses is negligibly small. The environmental actions can then be treated as quasi-static actions, see 9.4. Where dynamic effects need to be taken into account but remain limited in magnitude, the dynamic amplification of the global structural responses can be adequately approximated by the addition of an equivalent quasi-static action, see 9.8. If the dynamic effects exceed the limitations given in 9.8.1, a dynamic analysis is required.

9.3.2 Notation

The total quasi-static environmental action is denoted by E. Actions due to one of the environmental conditions separately are indicated by E with a subscript i for wind (E_i), w for waves (E_w) and c for current (E_c). Where actions due to waves and current are considered together, both subscripts w and c are attached (E_wc). An additional equivalent quasi-static action at the sea floor, taking into account dynamic effects, is denoted by D. The extreme value of any one of these actions is indicated by an additional subscript e, e.g. E_e, E_ie, E_wce, D_e.

9.4 Extreme quasi-static action due to wind, waves and current (E_e)

9.4.1 Procedure for determining E_e

Subclause 9.4 is directly applicable only to structures that have negligible dynamic amplification; the methods of this subclause cover the quasi-static part of the action only. Dynamic amplification shall be included if significant. Methods of treating dynamic effects are described in 9.8. E_e is the extreme quasi-static direct action, with a 100 year return period, that is exerted on the structure by the combined effect of wind, waves and currents in an extreme storm. E_e may therefore include the joint probability of the occurrence of extreme winds, waves and currents, both in magnitude and in direction.

One of three methods is normally used for defining an environment that generates the extreme direct action E_e and generally also the extreme action effect, caused by the combined extreme wind, wave and current conditions:

a) 100 year return period wave height (significant or individual) with associated wave period, wind and current velocities;

b) 100 year return period wave height and period combined with the 100 year return period wind speed and the 100 year return period current velocity, all determined by extrapolation of the individual parameters considered independently;

c) any reasonable combination of wave height and period, wind speed and current velocity that results in

   — the global extreme environmental action on the structure with a return period of 100 years, or

   — a relevant action effect (global response) of the structure (e.g. base shear or overturning moment) with a return period of 100 years.

These three methods are discussed in ISO 19901-1, which also describes the selection of appropriate parameters for determining the design conditions.

For structures for which the environmental action is dominated by waves, E_e can be estimated by method a) above. For other structures, other combinations of wind, wave and current can be more appropriate. In some cases, consideration of a 100 year extreme current velocity with an associated wave height, wave period(s) and wind can provide the best estimate of E_e.

In locations where joint probability information for the environmental conditions is not available, a conservative estimate of E_e can be determined through the sum of the environmental actions caused by the independent extreme values of wind velocities, wave heights and periods, and current velocities, assuming that they act simultaneously and in the same direction, method b) above. Use of this method results in Equation (9.4-1):

$$E_e = E_{ie} + E_{wce} \tag{9.4-1}$$

The most general approach for correctly estimating E_e due to combinations of wind, waves and current is via the calculation of the long-term statistics of global environmental actions (such as applied base shear or overturning moment), or action effects for a generic structure at the intended location, method c) above. The statistical distributions thus obtained represent response-based global environmental actions for the type of structure and location involved. They contain the most appropriate information on actions for in-place design situations. Based on these long-term statistical distribution(s), a particular combination of wind, wave and current parameters can be identified that is most likely to generate the 100 year extreme global environmental action(s), in conjunction with a corresponding partial action factor, $\gamma_{f,E}$, that provides adequate protection against failure under environmental actions in an extreme storm.

An acceptable procedure by which E_e may be calculated is given in 9.5 to 9.7.

In the absence of site-specific data, regional information in ISO 19901-1 can give an indication of the extreme environmental conditions in certain geographical areas. These values are only indicative (e.g. for use in conceptual design studies), unless stated otherwise; therefore, for the final design of a structure, the owner shall use site-specific data.

9.4.2 Direction of extreme wind, waves and current

In some locations, wave height will vary by direction due to either characteristic storm tracks or topographic features limiting the fetch. Similarly, tidal currents, general circulation currents or wind-driven currents can also have predominant directions. In such situations, different extreme wind, waves and/or current magnitudes for different approach directions may be used in design, provided that reliable data are available to derive these. However, the owner shall ensure that the overall safety level of the structure is not compromised by the use of such lower directional environmental conditions.

When directional data are used, directional sectors should generally not be smaller than 45°. The environmental conditions should be scaled up such that the most severe sector is no less severe than the omni-directional 100 year condition. The tolerance on orientation, or the as-installed orientation, of the structure at installation should be taken into account when determining the appropriate parameters.

In locations where reliable directional data are not available, the directions of the extreme wind, waves and current shall be assumed to coincide when determining E_e.

9.4.3 Extreme global actions

Total base shear and overturning moment are calculated by a vector summation of

a) local hydrodynamic drag and inertia actions due to waves and currents (see 9.5.2) integrated over the whole structure,

b) dynamic amplification of wave and current actions (see 9.8), and

c) actions on the structure and the topsides caused by wind (see 9.7).

The above pre-supposes that there is sufficient air gap and that the extreme wave crest does not impinge on the topsides. If this condition is not satisfied, actions associated with the wave crest impinging on the topsides shall be included in a).

Wave slamming and slapping actions due to water surface impact on structural components may be neglected, as they are only of local significance. Actions due to hydrodynamic lift may be neglected for space frame type structures because they are not correlated from member to member. Axial Froude-Krylov actions may also be neglected. The wave crest shall be positioned relative to the structure and the water depth shall be selected such that the total base shear and/or the overturning moment have their maximum values. It should be kept in mind that

— maximum base shear does not necessarily occur at the same wave position as maximum overturning moment,

— maximum base shear often occurs at minimum possible water depth (low tide with minimum storm surge), and

— maximum global action can be induced by the trough of a wave in a direction opposite to the direction in which the waves propagate, especially when an opposing current is present and the wave height is moderate.

Shallow water effects and limits on wave models shall be investigated where appropriate. Particularly in shallow water, the range of water depths between the maximum and minimum depth in the 100 year storm shall be investigated to determine the maximum global action.

9.4.4 Extreme local actions and action effects

The following shall be considered in the local design of components:

a) maximum local actions on, as well as maximum internal forces in, a component can occur for wave positions other than those causing the maximum global action, and maximum local forces in all components will not occur for the same wave position;

b) some components can see their maximum internal force due to global action in one direction, while others see their maxima due to the global action in another direction;

c) slamming actions on horizontal components and slapping actions on inclined components can occur when the water surface impacts on components at right angles to the component's surface, resulting in local actions perpendicular to the component.

9.4.5 Vortex induced vibrations (VIV)

All slender members shall be investigated for the possibility of in-line and/or cross-flow vibrations due to vortex shedding resulting from the flow of water or air past the member, as appropriate.

Where relevant, the impact that cross-flow vibrations can have on the hydrodynamic coefficients given in 9.5.2.3 shall be considered.

9.5 Extreme quasi-static action caused by waves only (E_{we}) or by waves and currents (E_{wce})

9.5.1 Procedure for determining E_{we} and E_{wce}

The procedure used to determine the extreme, deterministic, quasi-static global action exerted on the structure caused by waves alone or by waves and current is applicable only to a fixed structure that satisfies the following conditions:

— negligible distortion of the incident wave by the structure;

— negligible dynamic structural response.

The sequence of steps described below is shown graphically in Figure 9.5-1. The procedure, for a given wave direction, begins with the specification of the design wave height and period, storm water depth and current profile. If the action of waves only is to be calculated, the current velocity is put to zero with no change to the procedure.

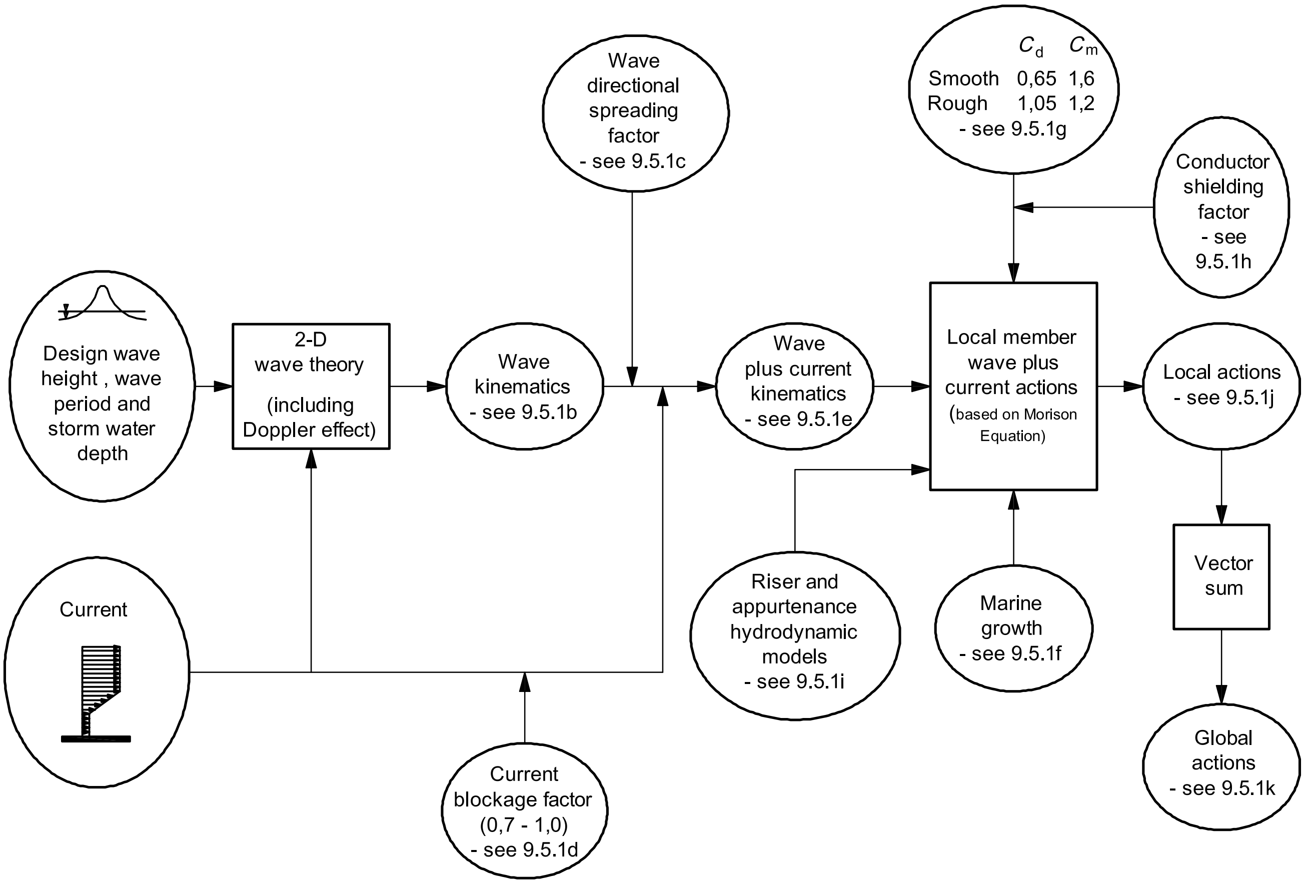


Figure 9.5-1 — Procedure for calculating the quasi-static action caused by wave plus current

The procedure for estimating the extreme quasi-static action caused by waves and current follows the steps a) to k).

a) Determine the intrinsic wave period, taking into account the Doppler effect of the current on the wave length and period, in accordance with ISO 19901-1.

b) Determine the two-dimensional wave kinematics for the specified wave height, storm water depth, and intrinsic period, using an appropriate wave theory; see ISO 19901-1.

c) The horizontal components of the wave induced water particle velocities and accelerations may be reduced by the wave directional spreading factor; see ISO 19901-1.

d) Determine the effective local current profile by multiplying the specified current profile by the current blockage factor; see 9.5.2.4 and A.9.5.2.4.

e) Determine the locally incident fluid velocities and accelerations for use in Morison's equation (see 9.5.2.1) by vectorially combining the effective local current profile from step d) after stretching it to the instantaneous water surface elevation (see ISO 19901-1) with the wave kinematics from step c).

f) Increase the member diameters to account for marine growth; see 9.5.2.2.

g) Determine drag and inertia coefficients for use in Morison’s equation in accordance with the relevant wave and current parameters, as well as member shape, size, orientation and roughness (including marine growth). Typical values of these coefficients are given in 9.5.2.3.

h) Drag and inertia coefficients for the conductor array may be reduced by the conductor shielding factor in 9.5.2.5.

i) Determine hydrodynamic models for actions on risers and appurtenances in accordance with 9.5.3.

j) Calculate local actions caused by waves and current for all structural components, conductors, risers, and appurtenances using Morison's equation, taking the requirements of steps e) to i) into account.

k) Calculate the extreme global action caused by waves and current as the vector sum of all the local actions from step j).

9.5.2 Models for hydrodynamic actions

9.5.2.1 Morison equation

The computation of the action on a cylindrical object (a member) caused by waves, currents or a combination of waves and currents depends on the ratio of the wave length to the member diameter. When this ratio is large (> 5), the member does not significantly modify the incident wave. The action can then be computed as the sum of a hydrodynamic drag action and a hydrodynamic inertia action, as given in Equation (9.5-1):

$$\boldsymbol{F} = \boldsymbol{F}_{\mathrm{d}} + \boldsymbol{F}_{\mathrm{i}} = C_{\mathrm{d}} \cdot \tfrac{1}{2}\rho_{\mathrm{w}} \cdot \boldsymbol{U} \cdot \left|\boldsymbol{U}\right| \cdot A + C_{\mathrm{m}} \cdot \rho_{\mathrm{w}} \cdot V \cdot \frac{\partial \boldsymbol{U}}{\partial t} \tag{9.5-1}$$

where

$\boldsymbol{F}$ is the local action vector per unit length acting normal to the axis of the member;

$\boldsymbol{F}_{\mathrm{d}}$ is the vector for the drag action per unit length acting normal to the axis of the member in the plane of the member axis and $\boldsymbol{U}$;

$\boldsymbol{F}_{\mathrm{i}}$ is the vector for the inertia action per unit length acting normal to the axis of the member in the plane of the member axis and $\frac{\partial \boldsymbol{U}}{\partial t}$;

C_{d} is the hydrodynamic drag coefficient;

ρ_{w} is the mass density of water;

A is the effective dimension of the cross-sectional area normal to the member axis per unit length (= D for circular cylinders);

V is the displaced volume of the member per unit length (= $\pi D^2/4$ for circular cylinders);

D is the effective diameter of a member (a circular cylinder), including marine growth;

$\boldsymbol{U}$ is the component of the local water particle velocity vector (due to waves and/or current) normal to the axis of the member;

$\left|\boldsymbol{U}\right|$ is the modulus (the absolute value) of $\boldsymbol{U}$;

C_{m} is the hydrodynamic inertia coefficient;

$\frac{\partial \boldsymbol{U}}{\partial t}$ is the component of the local water particle acceleration vector normal to the axis of the member.

The Morison equation, as stated here, ignores the contribution from the Froude-Krylov action associated with the convective acceleration component. This is not always appropriate for inertia dominated structures, small wave heights, steep waves or very large diameter components. The equation also excludes actions due to hydrodynamic lift, slamming or slapping actions and axial Froude-Krylov actions.

When the size of a structural object or component is sufficiently large compared to the wave length, the incident waves are scattered or diffracted. This “diffraction regime” is usually considered to occur when the component width exceeds a fifth of the incident wave length. In such cases, diffraction theory, which computes the pressures acting on the structure due to both the incident wave and the scattered wave, shall be used instead of the Morison equation to determine the local action caused by waves. If a structure has some components in the Morison regime and others in the diffraction regime, or a component is adjacent to another (component of a) structure that is in the diffraction regime, the effect of wave diffraction on the wave kinematics to be used for the Morison component should be considered.

NOTE Depending on its diameter, a free-standing caisson structure can be in the diffraction regime, particularly for the lower sea states associated with fatigue conditions.

9.5.2.2 Marine growth

The expected effect of marine growth on the hydrodynamic actions on the structure during its design service life shall be taken into account.

The cross-sectional dimensions of structural components, conductors, risers and appurtenances shall be increased to account for marine growth thickness. Components with circular cross-sections shall further be classified as either “smooth” or “rough”, depending on the amount and size of marine growth expected to have accumulated on them at the time of the loading event.

Structural elements can be considered hydrodynamically smooth if they are located either above the highest astronomical tide (HAT) or sufficiently deep below the lowest astronomical tide (LAT). In both these zones marine growth accumulation is usually small enough to ignore the effect of roughness. However, caution should be taken because it takes very little roughness to make the rough value of the C_{d} in 9.5.2.3 realistic. Site-specific data shall be used to reliably establish the extent of the hydrodynamically rough zones. Otherwise, the structural components shall be considered rough down to the sea floor.

9.5.2.3 Drag and inertia coefficients

For typical design situations, global hydrodynamic action on a structure can be calculated using Morison’s equation, with the values of the hydrodynamic coefficients for unshielded circular cylinders given in Table 9.5-1:

Table 9.5-1 — Typical values of hydrodynamic coeficients

Surface of component	C_{d}	C_{m}
smooth	0,65	1,6
rough	1,05	1,2

The values in Table 9.5-1 are appropriate for situations of a steady current with negligible waves and for large waves with $U_{\mathrm{mo}}T_{\mathrm{i}}/D > 30$, where

U_{mo} is the maximum horizontal water particle velocity at storm still water level under the wave crest from the two-dimensional wave kinematics theory;

T_{i} is the intrinsic wave period;

D is the diameter of the structure’s legs at storm still water level.

Additional information and guidance on hydrodynamic coefficients in a variety of circumstances is given in A.9.5.2.3.

For situations where waves are dominant but $U_{\mathrm{mo}}T_{\mathrm{i}}/D < 30$, C_{m} and C_{d} for nearly vertical members are modified by “wake encounter” and shall be specially determined.

NOTE Such situations can arise for a large diameter free-standing caisson structure in extreme seas or for ordinary structural components in the lower sea states that are considered in fatigue analyses.

9.5.2.4 Current blockage

The in-line current speed in the vicinity of a structure is reduced from the specified free stream value by blockage. The presence of the structure causes the incident flow to diverge; some of the flow goes around the structure rather than through it, and the in-line current speed within the structure is reduced.

Since global hydrodynamic action on the structure is determined by summing local hydrodynamic action contributions from Morison's equation, the appropriate local current velocity shall be used. Data sets obtained from measurements of the current speed in the vicinity of a structure will generally already include the effect of current blockage in the data. Therefore, current blockage factors should not be applied to such (extrapolated) data sets.

9.5.2.5 Conductor shielding factor

Depending upon the configuration of the structure and the number of well conductors, the hydrodynamic action on the conductors can be a significant portion of the total hydrodynamic action on the structure. If the conductors are closely spaced, the global hydrodynamic action on them can be reduced as a result of hydrodynamic shielding. To account for this, a shielding factor, k_{s}, may be applied to the drag and inertia coefficients for the conductor array. The shielding factor can be estimated from Figure 9.5-2, in which S is the centre-to-centre spacing of the conductors in the wave direction and D is the diameter of the conductors, including marine growth. This shielding factor is appropriate for either

a) steady current with negligible waves, or

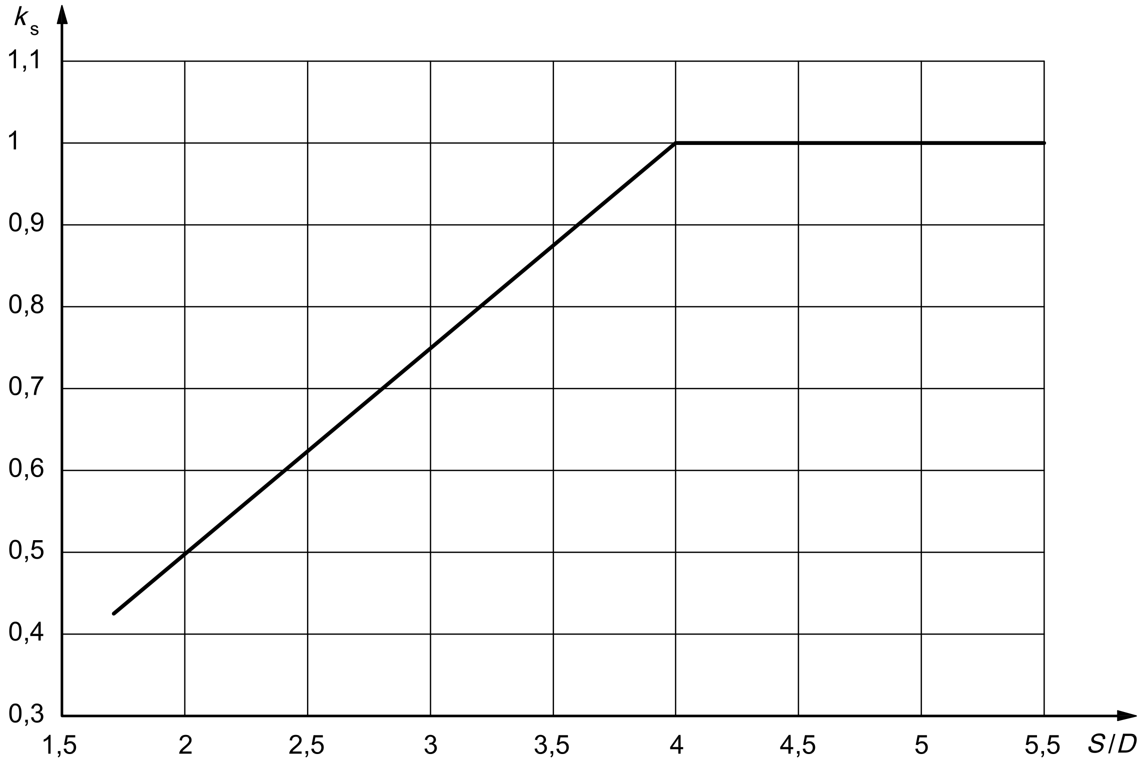
b) extreme waves, with $U_{\mathrm{mo}}T_{\mathrm{i}}/S > 5\,\pi$, where U_{mo} and T_{i} are defined in 9.5.2.3.

For less severe waves, with $U_{\mathrm{mo}}T_{\mathrm{i}}/S < 5\,\pi$, as in fatigue analyses, shielding shall not be invoked.

When the angle between the wave or current direction and the centre plane of a row (or column) of conductors is less than 22,5°, S is taken as the centre-to-centre spacing between the conductors in the row (or column). For angles between 22,5° and 67,5°, S can be approximated as the average of the centre-to-centre row spacing and the centre-to-centre column spacing. For a single row of conductors, shielding should not be included if the angle between the centre plane of the row and the wave or current direction is greater than 22,5°.

The shielding phenomenon in this subclause does not adequately address the situation of small diameter members adjacent to large diameter members, e.g. an external conductor at a monotower. The interaction between the small and the large diameter member in such situations shall be carefully considered; see also 9.11.

A possible increase in the added mass (and inertia actions) for closely spaced conductors shall be considered.



Key

S conductor spacing

D conductor diameter

k_s shielding factor

Figure 9.5-2 — Factor for reduction of hydrodynamic coefficients for conductors in array due to shielding as function of conductor spacing/diameter ratio

9.5.3 Hydrodynamic models for appurtenances

Appurtenances, such as boat landings, fenders or bumpers, walkways, stairways, grout lines, and anodes, shall be considered for inclusion in the hydrodynamic model of the structure. Depending upon the type and number of appurtenances these can significantly increase the global hydrodynamic action on the structure. Furthermore, hydrodynamic actions on some appurtenances can be important for local member design. Appurtenances are generally modelled by means of non-structural members that contribute equivalent hydrodynamic action. For some appurtenances, such as boat landings, the hydrodynamic action is highly dependent on wave direction due to shielding effects.

9.6 Actions caused by current

The current velocity and current profile as a function of depth shall be determined in accordance with ISO 19901-1.

Where current acts alone, the local action due to current can be calculated in accordance with the procedure in 9.5 (as far as relevant) by putting $\partial U/\partial t = 0$. Where current acts together with waves, the current velocity shall be stretched and vectorially added to the wave particle velocity before the total local action due to waves and current is calculated, as described in 9.5.

Where a global action caused by current only is to be determined, the extreme quasi-static global action, E_{ce}, shall be calculated as the vector sum of the above local current actions.

9.7 Actions caused by wind

9.7.1 General

The wind velocity, vertical wind profile and time averaging duration in relation to the dimensions and dynamic sensitivity of the structure's components shall be determined in accordance with ISO 19901-1. In special cases, dynamic response to wind action can be significant and shall be taken into account; see ISO 19901-1.

9.7.2 Determining actions caused by wind

For all angles of wind approach to the structure, wind actions on vertical cylindrical objects may be assumed to act in the direction of the wind. Actions on cylindrical objects that are not in a vertical attitude shall be calculated using appropriate formulae that take into account the direction of the wind in relation to the attitude of the object, in the same way as drag actions on submerged cylinders due to wave and current are calculated. Actions on walls of buildings and other flat surfaces that are not perpendicular to the direction of the wind shall also be calculated, using appropriate formulae that account for the skewness between the direction of the wind and the plane of the surface. Where appropriate, actions caused by wind shall be calculated with due account for increased exposure area and surface roughness due to ice. Local wind effects, such as pressure concentrations and internal pressures, should be considered by the designer where applicable.

The wind action on an object may generally be calculated using Equation (9.7-1). However, in accordance with the above observations due attention shall be given to which velocity (in magnitude and direction) and which area is to be used. Similarly, the direction of the resulting action vector shall be carefully considered. The basic relationship between the wind velocity and the wind action on an object is expressed by Equation (9.7-1) :

$$F = \frac{1}{2}\rho_a U_w{}^2 C_s A \qquad (9.7\text{-}1)$$

where

F is the wind action on the object;

ρ_a is the mass density of air (at standard temperature and pressure);

U_w is the wind speed;

C_s is the shape coefficient;

A is the area of the object.

In the absence of data indicating otherwise, the shape coefficients given in Table 9.7-1 are recommended for perpendicular wind approach angles with respect to each projected area.

Table 9.7-1 — Shape coefficients, C_s, for perpendicular wind approach angles

Component		shape coefficients C_s
Flat walls of buildings		1,50
Overall projected area of structure		1,00
Beams		1,50
Cylinders	Smooth, $Re > 5 \times 10^5$	0,65
	Smooth, $Re \leqslant 5 \times 10^5$	1,20
	Rough, all Re	1,05
	Covered with ice, all Re	1,20
Re Reynolds number		

Wind actions on downstream components can be reduced due to shielding by upstream components.

See A.9.7.2 for further information and guidance on the use of calculation method and coefficients.

The extreme quasi-static global action, E_{ie}, caused by wind shall be calculated as the vector sum of the above wind actions on all objects.

9.7.3 Wind actions determined from models

When wind actions are important for structural design, wind pressures and resulting local actions shall be determined from wind tunnel tests on a representative model, or from a computational model representing the structure, considering the range and variation of wind velocities. Computational models shall be validated against wind tunnel tests or full scale measurements of similar structures.

9.8 Equivalent quasi-static action representing dynamic response caused by extreme wave conditions

9.8.1 General

Actions caused by waves vary with time. Any structure subjected to these actions will therefore experience dynamic response to a greater or lesser degree. However, in many cases the dynamic response will remain rather limited in magnitude. In such cases and in accordance with ISO 19900, the global effects of dynamic response can be adequately approximated by the introduction of an equivalent quasi-static action, D, representing the overall dynamic action effect. The corresponding equivalent quasi-static action in design environmental conditions (D_e) is added to the total extreme quasi-static action due to wind, waves and current E_e.

In general, all (quasi-)static and dynamic actions are vectors. However, for the purposes of this International Standard and for the conditions under which an equivalent quasi-static action representing dynamic response may be used, only scalar values are required. The notations D_e, E_e and E_{wce}, as used in this International Standard, refer to the magnitudes of the corresponding vectors at the sea floor. Once determined, the equivalent extreme quasi-static action, D_e, is next distributed over the height of the structure in a set of horizontal actions, d_k, at elevations $k = 1, 2, \ldots\ldots, K$. The actions d_k represent the inertial actions caused by a global acceleration of the masses lumped at each elevation k. The actions d_k are then added to the corresponding actions due to E_e at the same elevation.

For the applicability of the approximate procedure described in 9.8, it is required that

a) the dynamic and the quasi-static responses be approximately in phase, and

b) the magnitude of D_e be considerably smaller than the magnitude of E_{wce}.

These requirements are considered to be satisfied if the following two conditions are met:

— the dynamic response is stiffness controlled, which may be assumed if the fundamental natural period of the structure is less than one-fifth of the peak period of the wave spectrum of the design sea state.

— the magnitude of D_e is less than one-half of the magnitude of E_{wce}.

In all other cases, a more detailed dynamic analysis shall be performed. Requirements for such an analysis are covered in Clause 12.

9.8.2 Equivalent quasi-static action (D_e) representing the dynamic response

The magnitude of the equivalent quasi-static action, D_e, represents the total global inertial action that the structure experiences at the time when the total global dynamic response is a maximum. Under the conditions specified in 9.8.1, the return period of the extreme global dynamic response to the environmental action is approximately the same as that of the extreme quasi-static response to E_{wce} alone.

The equivalent quasi-static action, D_e, is established through a global dynamic analysis in waves. D_e can be determined from the results of the dynamic analysis in one of two ways:

a) directly, by subtracting the extreme quasi-static base shear (which is equal to the extreme applied horizontal wave action on the structure) from the extreme dynamic base shear obtained from the analysis;

b) indirectly, by multiplying E_{wce} by a factor (k_{DAF} – 1), where the k_{DAF} is a dynamic amplification factor on the quasi-static base shear that is derived from the global dynamic analysis as the ratio of the extreme dynamic base shear to the extreme quasi-static base shear.

D_e subsequently needs to be distributed along the height of the structure to model the individual contributions of the total global inertial action that act at each mass point; see A.9.8.2.

9.8.3 Global dynamic analysis in waves

9.8.3.1 General

Subclause 9.8.3 describes the global dynamic analysis for the purpose of determining D_e in 9.8.2.

9.8.3.2 Dynamic analysis methods

The time varying actions caused by waves shall realistically model the sea surface and the frequency content of wave action in extreme wave conditions. This necessitates the use of random waves; the procedure described in 9.5 is only appropriate for quasi-static analysis. When there are significant non-linearities in modelling the environment, the structure, the fluid-structure interaction or the structure-foundation interaction, time-history methods are preferred. When linearization of hydrodynamic drag actions, inundation effects, and structure-foundation interaction can be justified, frequency domain methods generally provide a more transparent and computationally more efficient means of using random waves and may be used for the global dynamic analysis.

9.8.3.3 Design sea state

The random waves shall correspond with one or more wave spectra that are plausible representations of sea state(s) that produce the 100 year design action effect or, alternatively, the 100 year design wave condition defined in 9.4; see also 6.5.2. Consideration should be given to including currents in the global dynamic analysis as currents associated with the design sea state can affect dynamic response through the drag action in Morison’s Equation, see Equation (9.5-1). Wind fluctuations may generally be neglected for conventional fixed steel offshore structures, i.e. actions caused by wind can be considered to be static actions.

9.8.3.4 Hydrodynamic action on a member

Morison’s Equation may be used to compute actions on members of rigid structures. Its relative velocity form shall not be used for fixed structures. Hydrodynamic actions in phase with the structure’s acceleration are taken into account by the hydrodynamic added mass. Hydrodynamic damping may be taken into account by an equivalent viscous damping as described in 9.8.3.6.

9.8.3.5 Mass

The dynamic model of a fixed structure shall include appropriately modelled masses of

- the topsides,
- the structural steel in the structure,
- the appurtenances and conductors,
- the mass of marine growth expected to accumulate on the structure,
- the mass of water within submerged members, and

— the hydrodynamic added mass of submerged members, taking into account increased outside member diameters due to marine growth.

9.8.3.6 Damping

In lieu of an explicit determination of various damping contributions, an equivalent viscous damping model may be used. The equivalent viscous damping coefficient is sensitive to many effects, including structure type and layout, the severity of excitation and response, and the relationship of the natural period of the structure to the peak period of the design wave spectrum. In the absence of substantiating information for damping values for a specific structure, a damping coefficient of 2 % to 3 % of critical may be used for the global dynamic analyses in extreme wave conditions. For less severe sea states used for fatigue analyses, damping values less than 2 % are more appropriate.

9.8.3.7 Stiffness

A linear elastic stiffness model of the structure that includes the interaction between the structure and the foundation is normally adequate for the global dynamic analysis in waves.

9.9 Factored actions

9.9.1 General

In accordance with 7.6 and 7.8.1, each representative action shall be multiplied by a partial action factor, γ_f. The design action(s), F_d, for a particular design situation comprise one or more combinations of factored actions.

This clause relates to actions and partial action factors for strength design satisfying the ultimate limit state during extreme conditions for the in-place situation; see 7.2 and 7.6. The corresponding design actions are specified in 9.9.2, 9.9.3 and 9.10. Design actions for operating situations are also specified in 9.10.

Actions and partial action factors for strength design satisfying accidental limit states are given in Clause 10.

Actions and partial action factors for fatigue limit states are given in Clause 16; those for fatigue limit states of grouted and mechanical connections and clamps are given in Clause 15.

9.9.2 Factored permanent and variable actions

The design action for permanent and variable actions is given by Equation (9.9-1):

$$F_d = \gamma_{f,G1}\, G_1 + \gamma_{f,G2}\, G_2 + \gamma_{f,Q1}\, Q_1 + \gamma_{f,Q2}\, Q_2 \qquad (9.9\text{-}1)$$

where G_1, G_2, Q_1 and Q_2 are defined in 9.2.1 to 9.2.4, taking account of the requirements of 9.2.5 to 9.2.9.

Values for the partial action factors, γ_f, for various design situations are given in 9.10.

9.9.3 Factored extreme environmental action

The design action for extreme environmental action is given by Equation (9.9-2):

$$F_d = \gamma_{f,E}\,(E_e + \gamma_{f,D}\, D_e) \qquad (9.9\text{-}2)$$

where

E_e is the extreme quasi-static action due to wind, waves and current, defined in 9.4, taking account of the requirements of 9.5 to 9.7;

D_e is the equivalent quasi-static action representing the dynamic action effect defined in 9.8.1;

$\gamma_{f,E}$ is the partial action factor for E_e;

$\gamma_{f,D}$ is an additional partial action factor for D_e.

The value of $\gamma_{f,E}$ depends principally on

a) the target safety level represented by the exposure levels L1, L2 and L3, see 6.6,

b) the characteristics of the long-term environment at the offshore location of the structure, specifically the local climate of waves, current and wind at the geographical location (the wave climate usually being the most influential, in particular its sensitivity to return period), and

c) the geometrical and structural properties of the structure considered.

Additionally, the value will generally be affected by the method(s) used to derive the extreme wind, wave and current parameters and the corresponding extreme environmental action.

For steel space frame structures and a number of geographical areas, or regions within geographical areas, studies have been performed to determine appropriate values of $\gamma_{f,E}$. Some examples are presented in A.9.9.3.3; further information, where available, can be found in Annex H.

The value of $\gamma_{f,D}$ accounts for the uncertainty of D_e to represent the global dynamic response of the structure, as well as the variability of the influence of global dynamic response on individual components of the structure.

See A.9.9 for a further discussion on the partial action factors $\gamma_{f,E}$ and $\gamma_{f,D}$.

9.10 Design situations

9.10.1 General considerations on the ultimate limit state

The basic requirement for satisfying the ultimate limit state under design environmental conditions in the in-place situation for structures that respond predominantly quasi-statically in accordance with the conditions of 9.9.1 is that the structure possess an adequate strength to meet a minimum safety level commensurate with its exposure level; see 7.10.

This requirement can in principle be met in several ways, two of which are listed below and both of which are within the scope of this International Standard:

— by implicitly following a partial factor design format, which is the usual design and assessment procedure and which is described in 9.10.3;

— by explicitly demonstrating that a structure has a certain minimum reserve strength ratio (RSR), as discussed in 9.10.2.

For structures where dynamic response exceeds the limitations of 9.8.1 and a proper dynamic analysis is performed, the requirements for the ultimate limit state shall be explicitly considered.

9.10.2 Demonstrating sufficient RSR under environmental actions

A structure’s ultimate system strength may be determined by means of a suitable non-linear push-over analysis, from which the RSR can be calculated. If the RSR meets an appropriate criterion for

a) the relevant exposure level L1, L2 or L3, (see 6.6), and

b) the geographical location concerned,

the design satisfies the ultimate limit state for the situation being considered.

This approach represents a significant departure from conventional design practice and can be hampered by several difficulties. These difficulties include the need for special and advanced knowledge and experience of the designers, the limited relevant experience that is available with this approach, the fact that the calculated ultimate strength depends on the method and the details by which the assessment is carried out, and the lack of common agreement on the setting of appropriate RSR criteria. Consequently, the approach can only be adopted if it is approved by the owner and the regulator, where one exists; such approval shall include the methodology to be used and the RSR value to be achieved. For a discussion of the elements involved, see A.9.9.

Notwithstanding this, when these difficulties can be overcome and the owner, the designer and the regulator, where one exists, can reach agreement on all the aspects involved, the approach is fully in agreement with the general requirements of ISO 19900 and the requirements for fixed steel offshore structures in this International Standard.

9.10.3 Partial factor design format

9.10.3.1 General

For a quasi-statically responding structure, adequate proof of satisfying the ultimate limit state shall be deemed to be provided by the partial design format described in this subclause. Each member, joint and foundation component shall be checked for strength using the internal force (action effect) (S) due to the combinations of factored

a) permanent actions,

b) variable actions,

c) extreme quasi-static environmental actions, and

d) where relevant, equivalent quasi-static actions representing dynamic response caused by extreme wave conditions.

The combinations and factors are specified in 9.10.3.2.

9.10.3.2 Design actions for in-place situations

The general equation for determining the design action (F_{d}) for in-place situations is given in Equation (9.10-1), and the appropriate partial action factors for each design situation are given in Table 9.10-1:

$$F_{\mathrm{d}} = \gamma_{\mathrm{f,G1}} G_1 + \gamma_{\mathrm{f,G2}} G_2 + \gamma_{\mathrm{f,Q1}} Q_1 + \gamma_{\mathrm{f,Q2}} Q_2 + \gamma_{\mathrm{f,Eo}} (E_{\mathrm{o}} + \gamma_{\mathrm{f,D}} D_{\mathrm{o}}) + \gamma_{\mathrm{f,Ee}} (E_{\mathrm{e}} + \gamma_{\mathrm{f,D}} D_{\mathrm{e}}) \qquad (9.10\text{-}1)$$

where

G_1, G_2 are the permanent actions defined in 9.2;

Q_1, Q_2 are the variable actions defined in 9.2;

E_{o} is the environmental action due to the owner-defined operating wind, wave and current parameters;

D_{o} is the equivalent quasi-static action representing dynamic response in accordance with 9.8, but caused by the wave condition that corresponds with that for E_{o};

E_{e} is the extreme quasi-static action due to wind, waves and current as defined in 9.4 and taking account of the requirements of 9.5 to 9.7;

D_{e} is the equivalent quasi-static action representing dynamic response defined in 9.8.1;

$\gamma_{f,G1}$, $\gamma_{f,G2}$, $\gamma_{f,Q1}$, $\gamma_{f,Q2}$ are the partial action factors for the various permanent and variable actions discussed in 9.9 and for which values for different design situations are given in Table 9.10-1 (see A.9.10.3.2.1);

$\gamma_{f,Eo}$, $\gamma_{f,Ee}$ are partial action factors applied to the total quasi-static environmental action plus equivalent quasi-static action representing dynamic response for operating and extreme environmental conditions, respectively, and for which values for different design situations are given in Table 9.10-1;

$\gamma_{f,E}$, $\gamma_{f,D}$ are the partial action factors for the environmental actions discussed in 9.9 and for which appropriate values shall be determined by the owner.

Table 9.10-1 — Partial action factors for in-place situations and exposure level L1

Design situation	Partial action factors [a]					
	$\gamma_{f,G1}$	$\gamma_{f,G2}$	$\gamma_{f,Q1}$	$\gamma_{f,Q2}$	$\gamma_{f,Eo}$	$\gamma_{f,Ee}$
Permanent and variable actions only	1,3	1,3	1,5	1,5	0,0	0,0
Operating situation with corresponding wind, wave, and/or current conditions[b]	1,3	1,3	1,5	1,5	0,9 $\gamma_{f,E}$	0,0
Extreme conditions when the action effects due to permanent and variable actions are additive[c]	1,1	1,1	1,1	0,0	0,0	$\gamma_{f,E}$
Extreme conditions when the action effects due to permanent and variable actions oppose[d]	0,9	0,9	0,8	0,0	0,0	$\gamma_{f,E}$

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

[b] For this, check that G_2, Q_1 and Q_2 are the maximum values for each mode of operation.

[c] For this, check that G_1, G_2 and Q_1 include those parts of each mode of operation that can reasonably be present during extreme conditions.

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

9.11 Local hydrodynamic actions

Internal forces are due to both global actions transferred from the rest of the structure and local hydrodynamic actions on a member. In some cases, vortex induced vibrations add to the internal forces.

The calculation of the internal forces for the design of members and adjacent joints shall include these local actions with the partial action factor for environmental actions.

Transferred actions are due to the global environmental actions and the dynamic response of the entire structure. For stiffness controlled dynamic response, the latter will not be too severe and is taken into account as described in 9.8.

Local actions include not only the drag and inertia actions modelled by Morison's Equation, see Equation (9.5-1), but also actions due to hydrodynamic lift, axial Froude-Krylov actions, distributed weight and buoyancy effects. Horizontal members near storm still water level can experience vertical slamming actions as a wave passes. Actions due to hydrodynamic lift, slamming and slapping can dynamically excite individual members, thereby increasing internal forces. For smaller members adjacent to larger members, such as appurtenances supported by legs, the calculation of the kinematics used in Morison's equation should take account of the flow modification caused by the larger member.

The fraction of total internal forces due to locally generated actions is generally greater for members higher in the structure. Therefore, local actions due to hydrodynamic lift, slamming and slapping shall be considered in designing those members and adjacent joints. The maximum local internal forces can also occur at a different position of the wave crest relative to the structure centreline than those which cause the greatest global environmental action on the structure. For example, some members of conductor guide frames can experience their greatest internal forces due to vertical hydrodynamic actions, which generally peak when the wave crest is far from the structure centreline. Also, maximum internal forces in some members can occur for wind, wave and current directions other than those which cause maximum global environmental actions.

10 Accidental situations

This clause describes how hazards are treated and designed for in the context of this International Standard.

10.1 General

10.1.1 Hazards

Offshore structures are exposed to various hazards of greatly different types and having vastly different probabilities of occurrence. Hazards can be categorized in several ways, one of which is based on probability of occurrence. Using this criterion, hazards may be grouped into three main groups:

— **Group 1**

  hazards with a probability of occurring or being exceeded of the order of 10^{-2} per annum (return periods of the order of 100 years);

— **Group 2**

  hazards with a 10 to 100 times lower probability of occurring or being exceeded, i.e. probabilities of the order of 10^{-3} to 10^{-4} per annum (return periods of the order of 1 000 to 10 000 years);

— **Group 3**

  hazards with a probability of occurring or being exceeded markedly lower than 10^{-4} per annum (return periods well in excess of 10 000 years).

These probabilities (return periods) should be seen as an indication of the order of magnitude rather than as precise numbers, since accurate databases for such low probabilities of occurrence rarely exist.

The main hazards that are faced by an offshore structure include

— vessel (ship) collisions,

— dropped objects,

— fires and explosions, and

— abnormal environmental actions, including abnormal seismic actions.

Discussions of hazards and how these should be dealt with are often hampered by the use of various terminologies and concepts. In this International Standard, a hazard (3.24) is defined quite generally as the potential for human injury, damage to the environment, damage to property or a combination of these.

10.1.2 Designing for hazards

In accordance with ISO 19900, offshore structures and their structural components shall be designed to satisfy particular limit states. Each limit state is verified by defining a number of design situations and requiring that the associated action effects shall meet given design criteria. Design situations are classified into three categories:

a) persistent situations, with a duration similar to the design service life of the structure;

b) transient situations, with a much shorter duration and varying levels of intensity;

c) accidental situations, which are of short duration and low probability of occurrence.

NOTE In this International Standard, only designing for hazards for structures of exposure level L1 is quantified; specification of relevant design situations and criteria for exposure levels L2 and L3 is intended to be included in a future edition.

Designing for hazards of group 1 from exposure to design environmental conditions (wind, waves, current as well as earthquakes) is normally treated by the regular design process and incorporated in the verification of relevant limit states for persistent and transient design situations. Design wind, wave and current conditions and the corresponding extreme metocean parameters are described in ISO 19901-1, while the associated design actions (action effects) are given in Clause 9. The extreme level earthquake is described in ISO 19901-2 and the associated design actions (action effects) are given in Clause 11. Verification of the ultimate limit states (ULS) of structural components are described in Clauses 13, 14, 15 and 17. Verification of fatigue limit states (FLS) for the accumulation of action effects resulting from environmental conditions are described mainly in Clause 16 and partly in Clause 15.

Other hazards belonging to group 1 and not treated by the regular design process, as well as hazards belonging to group 2, are specially addressed by a requirement that the structure satisfy particular accidental limit states (ALS). Accidental limit states are verified through specified accidental situations, see 10.1.3 to 10.1.6.

Hazards falling into group 3 are sometimes referred to as residual accidents and may normally be neglected for design.

When a hazardous event occurs, the structure shall not be damaged disproportionately to the original cause. The intention of the ALS is to ensure that the structure can tolerate specified accidental situations and, if damage occurs, that it subsequently maintains structural integrity for a sufficient period under specified environmental conditions to enable evacuation to take place. This requirement is sometimes called the progressive collapse limit state (PLS).

Irrespective of a need for analysis and evaluation, due consideration shall always be given to providing robustness as specified in 7.9, both in the design of the structure and in the layout and arrangement of facilities and equipment on or in the topsides.

10.1.3 Accidental situations

Accidental situations, as introduced in ISO 19900 and used in this International Standard, relate to two types of hazards as follows.

— **Hazards associated with specially identified accidental events**

  These hazards belong to group 1 or group 2 and are not included in the regular design process.

— **Hazards associated with abnormal environmental actions**

  Abnormal environmental actions can occur due to the possible exposure to very rare and abnormally severe environmental conditions. The return period of the actions (action effects) exceeds the return period of the actions (action effects) due to design environmental conditions. The corresponding hazards fall into group 2.

The two types of hazards are different by nature. In principle, accidental events can in some cases be avoided by taking appropriate measures to eliminate the source of the event or by bypassing and overcoming its structural effects. In contrast with this, the possible occurrence of abnormal actions cannot be influenced by taking such measures.

An accidental situation normally comprises the occurrence of an identified accidental event or of abnormal environmental actions, in combination with expected concurrent operating conditions and associated permanent and variable actions.

In the occurrence of an accidental event or abnormal environmental actions, the structure can sustain damage. Such consequential damage creates a new situation, which is characterized by the after damage design situation. Subsequent to the hazardous event having occurred, the after damage design situation, which considers the structure's further behaviour, shall also be addressed. The structure shall consequently be designed following a two-stage procedure:

a) design the structure for the accidental situation involving the hazard considered;

b) after the hazardous event has occurred, check the after damage design situation in relation to specified environmental actions.

The second step is, of course, only necessary if the resistance of the structure is reduced by structural damage caused by the hazard; see also 10.1.6.

Identified accidental events are evaluated using accidental design situations (see 3.2 and 10.1.4).

Abnormal environmental actions are evaluated using abnormal design situations (see 3.1 and 10.1.5).

Consequential damage is evaluated using after damage design situations (see 3.3 and 10.1.6).

10.1.4 Identified accidental events

Typical examples of accidental events are

— collision from vessels (10.2),

— impact from dropped objects (10.3), and

— fire or explosion (10.4);

Designers can choose between avoiding a hazard (e.g. by taking special preventive measures such as operational restrictions), minimizing the (structural) consequences of the considered hazard (e.g. by implementing protective measures) or designing for the hazard.

If the design option is chosen, one or more accidental design situations shall be defined, see 10.1.3, and design requirements shall be established taking account of the operational conditions and the type, function and location of the structure. Structural components shall meet the strength requirements of Clauses 13, 14, 15 and 17. However, when checking accidental limit states (ALS) for accidental events, all partial action and resistance factors may be set to 1,0.

As accidental events are on the whole statistically independent, only one accidental event need be considered for a particular accidental design situation. If certain accidental events are not statistically independent, a corresponding accidental design situation or situations shall be specially considered.

10.1.5 Abnormal environmental actions

The appropriate return period for the abnormal design situation for abnormal actions due to wind, wave and current is dependent on the exposure level of the platform, the environmental climate (see ISO 19901-1), the acceptability of overall risk to a platform and its inhabitants, and the way in which that overall risk is apportioned to individual hazards facing the platform and its inhabitants. Additional guidance for certain geographical areas can be found in Annex H.

As discussed in 10.5, this abnormal design situation is normally less onerous than the extreme design situation, provided a sufficient air gap is available to avoid deck wave impingement on the topsides.

In lieu of other information, or of specific regional guidance in Annex H, this abnormal design situation may be based on a return period of 10 000 years for an exposure level L1 platform; see ISO 19901-1 for a discussion of such conditions. This return period is an indication of the order of magnitude rather than a precise number, since accurate databases for such small probabilities of exceedance rarely exist.

The abnormal design situation for abnormal earthquake actions may be based on the abnormal level earthquake according to ISO 19901-2. See Clause 11 for further seismic design considerations.

Structural components shall meet the strength requirements of Clauses 13, 14, 15 and 17. However, as for accidental events, when checking accidental limit states (ALS) for abnormal environmental actions, all partial action and resistance factors may be set to 1,0.

10.1.6 Damaged structures

10.1.6.1 Requirements for damage tolerance

For new structures, or those being assessed for compliance with this International Standard, the occurrence of damage following an accidental event or abnormal environmental actions can only be based on analysis. If a linear structural analysis of a corresponding design situation indicates no damage, then no further checks of the ALS for that situation are required. However, if damage is indicated, a reliable prediction of the nature and the extent of the damage normally requires the application of non-linear structural analysis methods.

The requirements of a platform having sustained damage are, to an extent, dependent on the environmental climate in the geographical area, regulatory requirements and the owner's preference. The possible requirements vary between the following:

— the platform shall remain intact for a period sufficient for all personnel to be evacuated (if manned at the time of the hazardous occurrence), for all process plant to be made safe and for all risk of pollution to be removed;

— the platform shall remain intact and without further damage for such time as it takes for repairs to be effected and for the platform to be made fit-for-purpose;

— the platform shall remain fit-for-purpose throughout.

In lieu of specific requirements, the after damage design situation shall be analysed using environmental conditions corresponding to actions (action effects) of a return period commensurate with twice a conservative assessment of the time required to effect suitable repairs (including inspection, design, fabrication and installation), by which the structure’s strength would be restored to the design strength; the minimum return period shall be one year. The specified partial action factors for the in-place situation (see Clause 9) shall be used. The strength of damaged components shall either be assessed following a rational approach, e.g. using the procedures of 7.7, or shall be neglected. Equations for the strength of certain dented members are given in Clause 13. Otherwise, the partial resistance factors and strength requirements for the in-place situation given in Clauses 13, 14, 15 and 17 shall be applied.

10.1.6.2 Assessment of structures following damage

For existing structures that have been damaged during service following an accidental event or abnormal environmental actions, the nature and extent of the actual structural damage shall be established by inspection in accordance with Clause 23. The modelling of the structure for the corresponding after damage design situation shall reflect the assessment of the actual damage as accurately as possible. Verification of the after damage design situation and the associated design criteria shall be in accordance with Clause 24.

Appropriate analysis combined with the information on the actual damage gathered by inspection shall determine any requirements for a shut-in and/or evacuation of the platform, as well as the need for any immediate (possibly temporary) repairs, while awaiting a decision and plan for the implementation of definitive repairs or for abandonment.

10.2 Vessel collisions

10.2.1 General

Vessel impact shall be addressed for structures with exposure levels L1 and L2.

The effect of a vessel impact shall be evaluated if the probability of collision is not negligible. In such an evaluation the nature of all vessel operations in the platform vicinity shall be taken into account. Such vessel operations can include vessels servicing the platform, fishing vessels working in the area and passing vessels.

Potential vessel impact on the structure's waterline members, risers, and external wells shall be considered. Barge bumpers, boat landings, and other external fendering may be used as protection.

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

— avoiding weak elements in the structure (particularly at joints),

— selecting materials with sufficient toughness, and

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

10.2.2 Collision events

In a rigorous impact analysis, if required, accidental design situations shall be established representing bow, stern and beam-on impacts on all exposed components. Evaluation of accidental design situations shall be in accordance with 10.1.

The collision events shall represent both a fairly frequent condition, during which the structure should only suffer insignificant damage, and a rare event where the emphasis is on avoiding a complete loss of integrity of the structure.

Two energy levels shall be considered:

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

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

The general requirements given in 10.1 apply to the accidental design situations for both energy levels. Level a) above represents a serviceability limit state to which the owner can set his own requirements based on practical and economical considerations, while level b) represents an ultimate limit state in which the structure is damaged but progressive collapse shall not occur.

In both cases, the analysis shall account for the vessel's mass, its added mass, orientation and velocity. Effective operational restrictions on vessel approach sectors can limit the exposure to impacts in some areas of the structure.

The vertical height of the impact zone shall be established, based on the dimensions and geometry of the structure and the vessel, and shall account for tidal ranges, operational sea state restrictions, vessel draught, and motions of the vessel.

10.2.3 Collision process

The energy absorbing mechanisms during the collision should be evaluated. Typically, local member denting, elastic and plastic deflection of the impacted member, global elastic and plastic response of the whole structure, and denting of the ship are the main mechanisms.

In a rigorous impact analysis, the collision actions should be evaluated based on a dynamic time simulation. The duration of the simulation should be sufficient to cover all relevant phases of the collision and the energy dissipation process.

10.3 Dropped objects

When evaluating the impact risk from dropped objects, the nature of all crane operations in the platform vicinity shall be taken into account. If the probability of impact is not negligible, relevant accidental design situations shall be defined and evaluated in accordance with 10.1. Depending on the consequences for the structural integrity of the structure, the need for a rigorous impact analysis shall be determined.

Irrespective of whether a rigorous analysis is required, robustness in relation to dropped objects should be incorporated into the design by indirect means such as

— avoiding weak elements in the structure (particularly at joints),

— selecting materials with sufficient toughness, and

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

10.4 Fires and explosions

Hydrocarbon pool fires on the sea surface and jet fires from ruptured risers can cause heating of structural components and hence degradation of their properties. Sources of hydrocarbons can include conductor or riser fracture, or spillage from the topsides following a process vessel rupture, while ignition sources can include radiation from oil burners and flares. ISO 13702 contains requirements and recommendations for fires and explosions. If the probability of exposure of the structure to fires is not negligible, relevant accidental design situations shall be defined and evaluated following the requirements of 10.1. Consequential damage resulting from such accidental design situations should be provided for in the structure design as required by 10.1.

Generally, conventional steel-framed structures with relatively small topsides that do not have enclosed compartments containing flammable fluids or ignition sources do not require design against explosion. However, if explosion studies of the topsides indicate significant or unusual actions on the structure or specific support requirements to ensure that the topsides integrity is maintained, then such actions or support requirements shall be provided for in the structure design in accordance with 10.1.

NOTE ISO 19901-3:[2] is to contain specific requirements for topsides structures for fires and explosions.

10.5 Abnormal environmental actions

Abnormal design situations for abnormal environmental actions due to wind, wave, current and, where applicable, earthquake, shall be defined and evaluated following the requirements of 10.1.

Abnormal environmental actions due to wind, wave and current, with a return period in accordance with 10.1.5, shall be determined and used to check structural integrity with all partial action and resistance factors set to 1,0. Normally, this abnormal design situation is less onerous than the design situation for the extreme environmental actions in accordance with Clause 9, provided that a sufficient air gap is available to avoid deck wave impingement on the topsides.

Table 10.5-1 compares the requirements for extreme and abnormal environmental actions.

Table 10.5-1 — Comparison of extreme and abnormal environmental action requirements

Requirement	Situation	
	Extreme environmental actions	Abnormal environmental actions
Governing clause for actions	Clause 9	Clause 10
Limit state	ULS	ALS
Return period	100 years	See 10.1.5, default 10 000 years
Partial action factor	See Clause 9, default 1,35	1,0
Partial resistance factors	See Clauses 13, 14, 15, 17; generally 1,05 to 1,25 but up to 2,0	1,0
Wave crest height	Associated with 100 year return event	Associated with abnormal environmental event

Where applicable, abnormal environmental actions and action effects due to the abnormal level earthquake shall also be determined; see Clause 11 for further seismic design considerations.

11 Seismic design considerations

11.1 General

The procedures in ISO 19901-2 for design against seismic events shall be followed. This clause complements ISO 19901-2 by presenting

a) the design actions, combinations of actions and action effects resulting from ground motions, and

b) the requirements for a fixed steel offshore structure subjected to seismic actions.

As outlined in ISO 19901-2, a two-level seismic design procedure shall be followed. The structure shall be designed to the ultimate limit state (ULS) for strength and stiffness when subjected to an extreme level earthquake (ELE), from which it should sustain little or no damage. The structure is next checked when subjected to an abnormal level earthquake (ALE) to ensure that it meets reserve strength and energy dissipation requirements. The structure may sustain considerable damage from an ALE, but structural failures causing loss of life and/or major environmental damage shall not be expected to occur.

11.2 Seismic design procedure

ISO 19901-2 gives alternative procedures for determining seismic actions and alternative methods of evaluation of seismic activity. The selection of the procedure and the method of evaluation of activity depend on a structure's seismic risk category (SRC). The SRC according to ISO 19901-2 depends on the platform's exposure level and the seismic zone in which it stands. The ISO 19901-2 requirements are the following:

a) to determine seismic actions using either the simplified seismic action procedure or the detailed seismic action procedure, as specified in ISO 19901-2;

b) evaluate seismic activity and the associated response spectra for the design of a structure against excitation of its base by ground motions using either ISO maps, regional maps or a site-specific seismic hazard analysis, as specified in ISO 19901-2;

c) demonstrate the ALE performance of the structure, which can require a non-linear analysis.

Specific recommendations for the ductile design of fixed steel offshore structures, dependent on the structure's SRC as given in Table 11.2-1, are presented in 11.4.

Table 11.2-1 — Applicability of ductile design recommendations

SRC	Recommendations for ductile design
1	Not applicable
2	Optional
3	Recommended
4	Recommended

11.3 Seismic reserve capacity factor

Both simplified and detailed seismic action procedures require an estimate of the seismic reserve capacity factor, C_r. This factor represents a structure's ability to sustain ground motions due to earthquakes beyond the level of ELE. It is defined as the ratio of spectral acceleration which causes structural collapse or catastrophic system failure to the ELE spectral acceleration. For fixed steel offshore structures, the representative value of C_r may be estimated from the general characteristics of a structure's design in accordance with Table 11.3-1:

Table 11.3-1 — Representative values of seismic reserve capacity factor, C_r

Characteristics of structure design	C_r
The recommendations for ductile design according to 11.4 are followed and a non-linear static pushover analysis according to 11.6.3 is performed to verify the global performance of the structure under ALE conditions.	2,80
The recommendations for ductile design according to 11.4 are followed, but a non-linear static pushover analysis to verify ALE performance is not performed.	2,00
The structure has a minimum of three legs and a bracing pattern consisting of leg-to-leg diagonals with horizontals, or X-braces without horizontals. The slenderness ratio ($K{\cdot}L/r$) of primary bracing in vertical frames is limited to no more than 80 [with the corresponding column slenderness parameter λ not exceeding $80/\pi \times \sqrt{(f_{yc}/E)}$, see Clause 13] and $f_y\, D/E\, t \leqslant 0{,}069$. For X-bracing in vertical frames the same restrictions apply, where the length L to be used depends on the loading pattern of the X-bracing. A non-linear analysis to verify ALE performance is not performed.	1,40
If none of the above characterizations apply.	1,10

Where the values of C_r according to Table 11.3-1 are not adopted, previous experience may be used to assume a value. In such cases, both of the following conditions shall apply.

— If the simplified seismic action procedure is followed, the assumed value of C_r shall not exceed 2,8 for L1 platforms, 2,4 for L2 platforms or 2,0 for L3 platforms;

— A non-linear time-history analysis in accordance with 11.6.4 shall be performed to ensure survival in the ALE event. As an alternative, a static pushover analysis in accordance with 11.6.3 may be performed to confirm that C_r is equal to or higher than that assumed.

11.4 Recommendations for ductile design

In seismically active areas, platform response to rare, intense earthquake motions can involve inelastic actions, and structural damage can occur. The provisions of this clause are intended to ensure that structure-foundation systems planned for such areas remain stable in the event of a rare, intense earthquake at the site. This can be achieved by providing sufficient system redundancy such that load redistribution and inelastic deformation will occur before collapse and by minimizing abrupt changes in stiffness in the vertical configuration of the structure. Adequate ductility can be demonstrated by adhering to the design practices outlined below or by non-linear analysis, where applicable.

The number of legs is very important for the reserve strength and energy dissipation capacity of a structure (sometimes also termed its ductility) and hence for the values of the seismic reserve capacity factor C_r given in 11.3. A structure with a minimum of three legs may be assumed to have a C_r value of 1,40 if the associated requirements given in 11.3 are met. A structure may be assumed to have a C_r value greater than or equal to 2,0 only if the structure-foundation system meets all of the requirements listed below.

— The structure shall have eight or more legs supported by piles.

— The piles shall be founded in competent soils that are not susceptible to liquefaction during the ELE and the ALE events.

— The legs of the structure, including any enclosed piles, shall meet the requirements of 11.5.1 using twice the design environmental action E during the ELE event.

— The vertical framing transmitting shear forces between horizontal frames shall consist of X-braces, or single (leg-to-leg) diagonals, and shall be arranged so that the shear between horizontal frames is carried by braces in both tension and compression (see Figure 11.4-1). K-bracing should not be used.

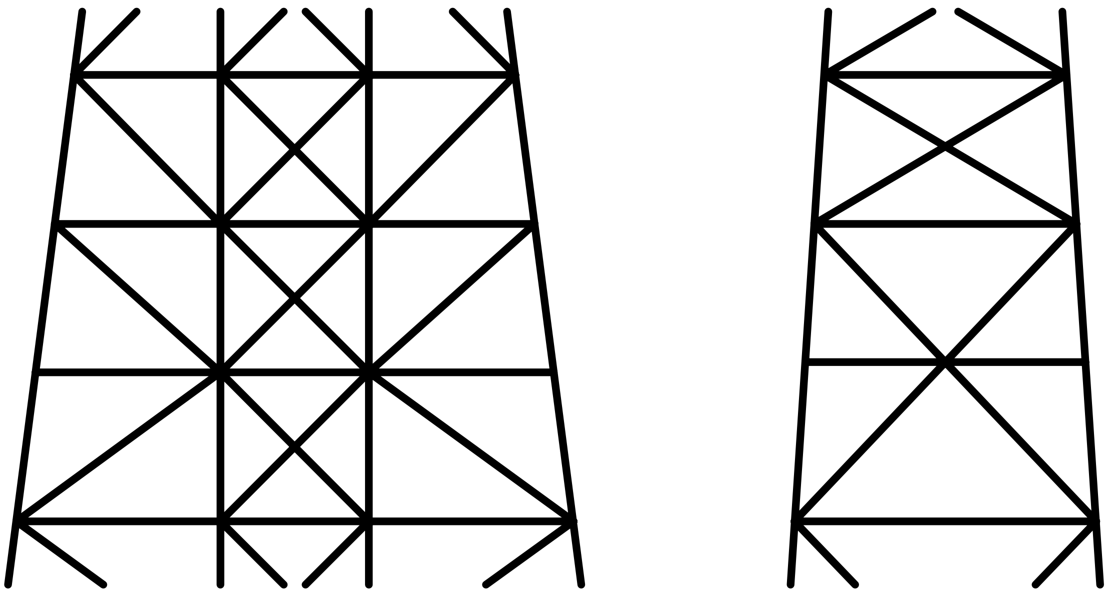


Figure 11.4-1 — Recommendations for vertical frame configurations for seismic conditions

— Horizontal members shall be provided between all adjacent legs at horizontal framing levels in vertical frames (see Figure 11.4-1) and these horizontal members shall have sufficient strength in compression to support the redistribution of actions resulting from any buckling of adjacent diagonal braces.

— The slenderness ratio ($K \cdot L/r$) of primary bracing in vertical frames shall be limited to no more than 80 (with the corresponding column slenderness parameter λ not exceeding $80/\pi \times \sqrt{(f_{yc}/E)}$, see Clause 13) and $f_y D/E \cdot t \leqslant 0{,}069$.

— All non-tubular members at connections in vertical frames shall have greater local buckling strength than global buckling strength, can develop fully plastic behaviour (i.e. are compact sections) and meet the requirements of 11.5.1 using twice the design environmental action E during the ELE event;

— All joints for primary structural members in the structure shall be sized to meet the minimum strength requirements given in 14.2.3. In lieu of this requirement, joint strengths may be verified by time-history analyses simulating the ALE event following the requirements of 11.6.4.

11.5 ELE requirements

11.5.1 Partial action factors

Each member, joint and foundation component shall be checked for strength using the internal force (action effect), S, resulting from the design action, F_d, calculated using Equations (11.5-1) and (11.5-2):

$$F_d = 1{,}1\ G_1 + 1{,}1\ G_2 + 1{,}1\ Q_1 + 0{,}9\ E \tag{11.5-1}$$

where E is the inertia action induced by the ELE ground-motion and determined using dynamic analysis procedures such as response spectrum analysis or time-history analysis. G_1, G_2 and Q_1 are defined in 9.2.1 to 9.2.3 and shall include actions that are likely to be present during an earthquake.

When contributions to the internal forces due to weight oppose the inertia actions due to the earthquake, the partial action factors for permanent and variable actions shall be reduced such that:

$$F_d = 0{,}9\ G_1 + 0{,}9\ G_2 + 0{,}8\ Q_1 + 0{,}9\ E \tag{11.5-2}$$

where G_1, G_2 and Q_1 shall include only actions that are reasonably certain to be present during an earthquake.

11.5.2 ELE structural and foundation modelling

The mass used in the dynamic analysis shall consist of the mass of the structure associated with

— the permanent actions G_1 and G_2,

— 75 % of the variable actions Q_1,

— the mass of entrapped water, and

— the added mass.

The added mass may be estimated as the mass of the displaced water for motion transverse to the longitudinal axis of individual structural members and of appurtenances. For motions along the longitudinal axis of the structural members and appurtenances, the added mass may be neglected.

The structural model shall include the three-dimensional distribution of the stiffness and mass of the structure. Joints in the model may be treated as rigid. Asymmetry in the distribution of the stiffness and mass of the structure can lead to significant torsion and should be considered in design.

In computing the dynamic characteristics of braced, pile-supported fixed steel offshore structures, a modal damping ratio of up to 5 % of critical may be used in the dynamic analysis of the ELE event. Additional damping, including hydrodynamic or soil induced damping, shall be substantiated by special studies.

Pile-soil performance and pile design requirements should be determined on the basis of studies that consider the design actions, installation procedures, cyclic and strain rate effects on soil properties, and characteristics of soils as appropriate for the axial or lateral capacity algorithm being used. When an equivalent linear foundation model is used, the stiffness of the pile foundation should be compatible with the expected level of non-linearity in the axial and lateral foundation response. See ISO 19901-4 for guidance.

The foundation stiffness can have a large effect on the natural period(s) of the structure, hence upper and lower bound foundation stiffness values shall be considered to evaluate the sensitivity to a range of possible natural periods.

For the design of piles for the ELE event, a partial resistance factor of 1,25 shall be used to determine the axial pile capacity (see 17.3.4) and a partial resistance factor for the p-y curves of 1,0 shall be used to determine the lateral pile performance (see 17.8).

11.6 ALE requirements

11.6.1 General

The structure-foundation system shall be analysed to demonstrate an ability to withstand the rare, intense ALE earthquake. This analysis shall establish that

a) the structure does not globally collapse during the earthquake, and

b) the structural integrity of the topsides is maintained.

The characteristics of the rare, intense ALE event shall be developed in accordance with ISO 19901-2. The stability of the structure-foundation system during the ALE event shall be demonstrated by analytical procedures that are rational and reasonably representative of the expected response of the structural and soil components to intense ground shaking. Performance requirements of the structure-foundation system shall be in accordance with ISO 19901-2.

The designer should develop a thorough insight into the performance of the structure and its foundation during the ALE event. The expected non-linear effects, including material yielding, buckling of structural components and pile failures, shall be adequately modelled and captured. The time-history method of analysis is recommended; however, a static pushover analysis may be used. While an analysis for the ELE event focuses on internal forces and stresses, the focus of an analysis for the ALE event is on strains and displacements.

11.6.2 ALE structural and foundation modelling

Analysis of the ALE event should be based on the most realistic estimate of values of parameters such as steel yield strength, member slenderness, member strength and soil strength used for determining the axial capacity and the lateral performance of piles. The partial resistance factors for axial capacity and lateral pile performance under ALE conditions shall be 1,0. Models of the structural and soil elements should include, as appropriate, the representative degradation of strength and stiffness under abnormal action reversals, the interaction of axial forces and bending moments in structural members, hydrostatic pressures, local inertial effects caused by members vibrating out of plane, and the P–Δ effect of earthquake actions.

NOTE See ISO 19901-4 for general guidance on foundation modelling.

In an analysis of the ALE event, the most realistic estimate of the yield strength of the material and of the non-linear behaviour of the joints (including any design strength bias) should be used. Local joint deformations greater than 5 % of the chord diameter shall be specifically investigated; otherwise, the joint should be assumed to fracture at this deformation level.

11.6.3 Non-linear static pushover analysis

The objective of a static pushover analysis is to verify that the seismic reserve capacity factor, C_r, of the structure as designed is greater than that initially estimated for design. The actions used in a static pushover analysis should represent the pattern of ALE seismic actions on the structure and foundation. Action patterns in a pushover analysis may be constructed to match the shear and moment distributions determined from an ALE response spectrum analysis along the height of the structure. Pushover analyses should be performed in several directions, as follows, in order to identify the structure's weakest direction:

— with the pattern of seismic actions aligned with the longitudinal (end-on) axis of the structure;

— with the pattern of seismic actions aligned with the transverse (broadside) axis of the structure;

— with the pattern of seismic actions aligned with one or more diagonal axes of the structure.

Diagonal direction(s) can be the weakest direction(s), especially with regard to foundation performance.

Yielding of structural members or piles shall not occur at global action levels lower than or equal to the global ELE action, F_{ELE} (Figure 11.6-1). The seismic reserve capacity factor, C_r, shall be the smallest value computed among all pushover analyses (i.e. weakest direction). C_r may be estimated using Equations (11.6-1) and (11.6-2) from the global seismic action-deformation curve obtained in a static pushover analysis, e.g. from global shear vs. deck displacement (see Figure 11.6-1):

$$C_r = C_{sr}\, C_{dr} \tag{11.6-1}$$

where C_{sr} is a factor corresponding to the strengthening regime of the action-deformation curve, estimated as

$$C_{sr} = \Delta_u / \Delta_{ELE} \tag{11.6-2}$$

where Δ_{ELE} is the deformation caused by the global ELE action F_{ELE} and Δ_u is the deformation corresponding to F_u, the ultimate action where the slope of the action-deformation curve becomes negative, see Figure 11.6-1.

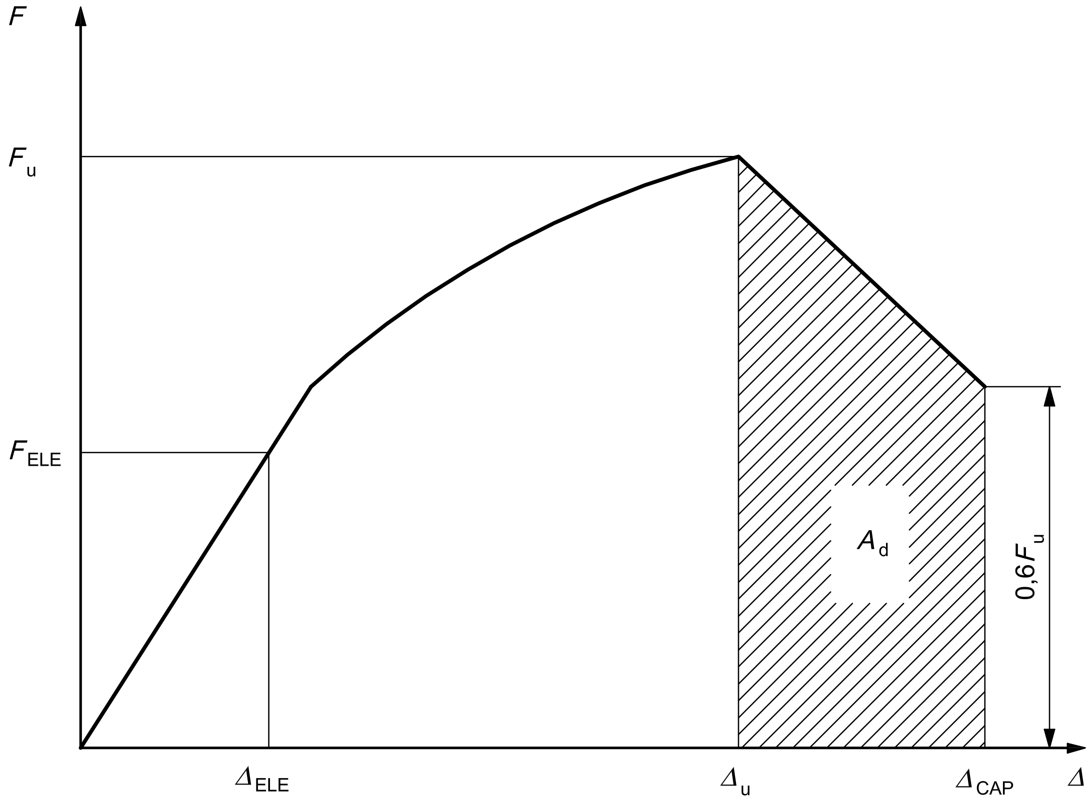


Figure 11.6-1 — Seismic action–deformation curve

C_{dr} is a factor corresponding to the degrading regime of the action-deformation curve. It is a measure of the energy dissipation capacity of the structure beyond the ultimate seismic action and the corresponding deformation. C_{dr} is estimated using Equation (11.6-3):

$$C_{dr} = \sqrt{1 + \frac{A_d}{F_u\, \Delta_u}} \tag{11.6-3}$$

A_d is the area under the action-deformation curve starting from Δ_u and ending with Δ_{CAP}. For the purpose of a non-linear static pushover analysis, the deformation capacity shall be assumed to be the deformation where the global action falls to 60 % of F_u.

NOTE The deformation to be used is a displacement of the deck (location of largest mass). As ratios of deformations and areas are used, the exact elevation within the deck is unlikely to have any effect. It is expected that the non-linearity will be confined to the structure below the deck and that the seismic action–deformation curves for different deck elevations will be similar.

The above determination of C_r presupposes that the primary sources of degradation of the global resistance have been properly modelled in the static pushover analysis to represent the structure's weakest direction, e.g. soil degradation, buckling of compression members, local buckling of members due to rotations at the member end (reducing the plastic moment capacity) and pushover direction.

Alternatively, Δ_u shall be defined as the deformation where the slope of the action–deformation curve is reduced to 5 % of the initial elastic slope, with C_{dr} assumed to be equal to 1,0.

11.6.4 Time-history analysis

The objective of a non-linear time-history analysis is to demonstrate that the structure can be expected to sustain the ALE seismic event without collapse and without major structural failure of the topsides. It shall be demonstrated that the structure-foundation system is expected to remain stable under the deformations induced by the ALE ground motions. The structure-foundation system shall be considered unstable when the deformations and degradation are severe enough to cause collapse under the influence of applicable permanent and variable actions.

The response of the structure-foundation shall be determined by a minimum of four sets of ground motion records characterizing the likely intensity, frequency content and duration of the ALE event. If seven or more time-history records are used, global structural survival shall be demonstrated in half or more of the time-history analyses. If fewer than seven time-history analyses are used, global survival shall be demonstrated in at least four time-history analyses.

11.7 Topsides appurtenances and equipment

Topsides design accelerations shall include the effects of the global dynamic response of the structure and, if appropriate, of the local dynamic response of the topsides and appurtenance itself.

Topsides equipment, appurtenances and piping which are both small (less than 5 % of the topsides weight) and stiff (local natural period less than half of the three-dimensional global 5^{th} mode) may be designed quasi-statically to resist peak deck accelerations from modal or time-history analyses of the overall structure.

It is recommended that general deck or topsides response spectra for design of major topsides equipment be obtained from time-history analyses of the complete structure. General deck or topsides spectra shall account for the variability in seismic responses of different locations on the deck to properly envelope horizontal, vertical and torsional motions of the deck. For a tall or seismically sensitive topsides subsystem (major equipment or module, e.g. drilling system, living quarters, flare), more specific response spectra can be developed, based on the motion of some support location point. Flexibility between that chosen support location and the subsystem shall be considered in order to properly capture the dynamic characteristics of the subsystem in an uncoupled analysis. A coupled analysis may be used, but care shall be exercised when coupling exists between the subsystem and the global modes. When coupling occurs, the sensitivity to modelling assumptions shall be evaluated.

The topsides response spectra should be the median values from at least four time-history analyses. Direct spectra-to-spectra generation techniques may also be used, however, such methods should be calibrated against the time-history method. The topsides and other equipment specific response spectra shall be broadened to account for uncertainties in structure frequencies and soil-structure interaction.

Seismic actions on major topsides equipment, piping and appurtenances should be derived by dynamic analysis using either

a) an uncoupled analysis with deck-level floor response spectra as input, or

b) a coupled analysis that properly includes a simple dynamic model of the relevant part of the topsides or the appurtenance in the global structural model (beware of modelling uncertainty and coupled interaction when including major equipment in dynamic structural models).

Equipment, piping, and other topsides appurtenances shall be designed and supported such that induced ELE actions can be resisted. Induced ELE displacements and deformations of the topsides shall be limited or designed against to avoid damage to the equipment, piping, appurtenances and supporting structures.

Drilling and well servicing structures shall be designed for ELE actions, using the response spectrum or time-history methods of analysis. It is important that these movable structures and their associated setback and pipe-rack tubulars are tied down or restrained at all times, except when the structures are being moved.

Safety critical systems and structures on or in the topsides shall be designed such that they are functional during and after the ALE event. Hazardous systems shall be designed such that they do not fail catastrophically (e.g. rupture) during the ALE event. In lieu of performing an ALE analysis of deck-supported structures, topsides equipment and equipment tie-downs, they shall be designed with an increased partial action factor on E of 1,15 rather than 0,9 in Equations (11.5-1) and (11.5-2). This partial action factor increase is only applicable if the ratio of ALE to ELE ground motions is less than 2,0. In areas where the ALE to ELE ratio exceeds 2,0, an additional increase in partial factor should be considered.

The use of walk-down techniques which are to be described in ISO 19901-3[2] are recommended for ensuring all necessary equipment is adequately supported for seismic conditions.

12 Structural modelling and analysis

12.1 Purpose of analysis

Structural analysis is the process of determining the action effects in a structure, or part thereof, in response to a given set of actions. It is a necessary step in demonstrating that a fixed steel offshore structure is able to satisfy the general requirements laid down in ISO 19900. Action effects required for the design of fixed steel offshore structures typically include the following:

— internal section forces, which shall not exceed the strength of the section;

— displacements and vibrations, which shall be within acceptable limits for operation of the structure;

— support reactions, from which the required foundation capacity can be determined.

This clause presents the requirements for structural analyses necessary for the design of fixed steel offshore structures for the following design situations:

a) pre-service situations including fabrication, loadout, transportation and installation (for which the design actions are given in Clause 8);

b) in-place situations, including

   — permanent, variable and environmental actions (for which the design actions are given in Clause 9), but excluding fatigue, which is dealt with in full in Clause 16,

   — accidental situations (Clause 10), and

   — seismic events (Clause 11);

c) removal situations (Clause 8);

d) structure reuse (Clause 25).

Structural analysis shall be based on actions and partial action factors specified in Clauses 8 to 11. Action effects calculated by structural analysis shall be used as input to the design or assessment process for components as described in Clauses 13 to 17.

12.2 Analysis principles

12.2.1 Extent of analysis

Structural analyses shall be performed to provide action effects suitable for checking the structure for all relevant design situations.

The number, types and extent of analyses to be performed shall cover all stages of the lifetime of the structure, i.e. pre-service, in-place and removal. However, if it can be demonstrated that particular stages in the design service life of a component of the structure do not govern its design, such stages need not be analysed for that component.

Requirements for typical analyses to be performed for a fixed steel structure and its components are given in 12.4. Complex or unusual structures can require other forms of analysis to determine all significant action effects in the structure.

Actions shall be applied simultaneously, if they can coexist, and shall include partial action factors, in accordance with Clauses 8 to 11, for each design situation being checked.

12.2.2 Calculation methods

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

Non-linear FEA is appropriate for the analysis of foundation components and may also be used to estimate ultimate strengths of structural components, and hence RSRs.

12.3 Modelling

12.3.1 General

This subclause provides guidance on the modelling of a structure, its structural components, their interactions and support conditions for the purpose of structural analysis.

Structural analyses can normally be limited to the structure being considered. However, where adjacent structures interact with the structure, the interaction shall be included in the analysis, to the extent required to predict the action effects accurately.

Due consideration shall be given to the interaction between primary structure, secondary structure, and the topsides structure (see 12.3.5).

Consideration shall further be given to any imperfections due to fabrication and to accidental damage.

12.3.2 Level of accuracy

Material properties and geometrical tolerances are defined in Clause 19 and Annex G. Where a particular component does not conform to these tolerances, the effect of the non-conformities on structural behaviour shall be evaluated. The effect of tolerances shall also be incorporated into the analysis where the structural design is particularly sensitive to their magnitude. The need for non-linear analysis to account for non-conformities shall be evaluated.

Where finite element analysis is performed, consideration shall be given to the type and accuracy of the element formulations, to ensure that sufficient elements are used to predict the structural behaviour, particularly in areas where the action effects change rapidly.

12.3.3 Geometrical definition for framed structures

12.3.3.1 General

Dimensions used in structural analysis calculations shall represent the structure as accurately as necessary to produce reliable values of the action effects.

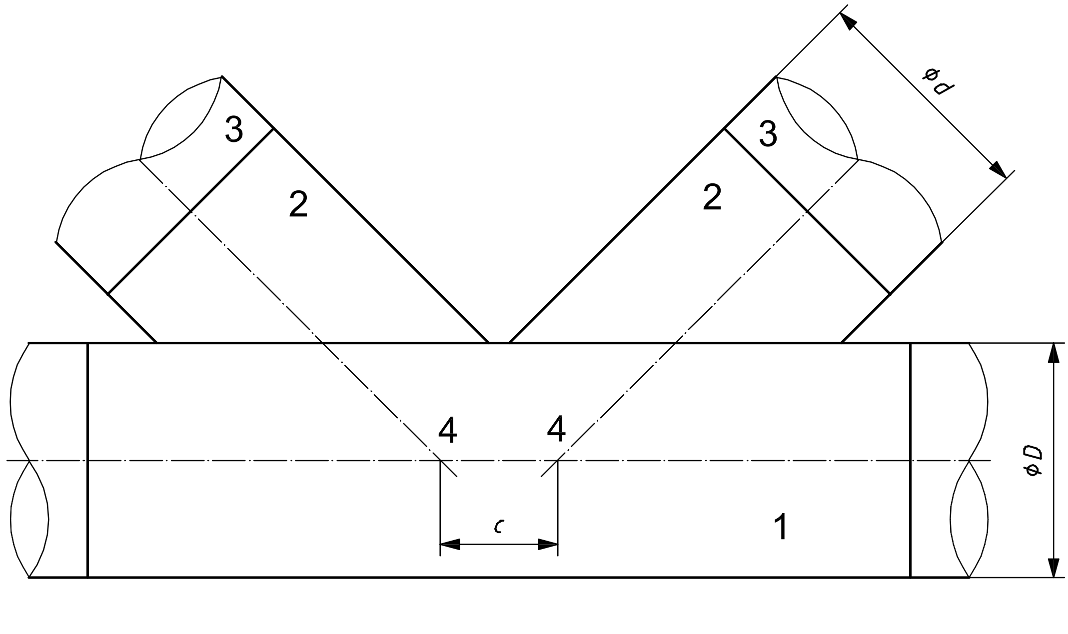
A space frame model shall be created to describe the three-dimensional geometry of the structure. The model is usually based on nominal sizes and dimensions. Where as-built dimensions differ from nominal sizes by more than the permissible tolerances set out in Annex G, the effect of this dimensional mismatch shall be assessed and incorporated into the analysis if appropriate.

The dimensions used in structural analysis calculations shall produce reliable, conservative values of the structural resistance and action effects. Stiffness and strength assessment shall normally be based on nominal sizes and dimensions. Mass, weight and hydrodynamic action calculations shall account for corrosion allowances (if any) and for the presence of marine growth.

A space frame model consists of members that are connected at nodes. The continuous member at a connection point (a node) is called the chord, members that end or are interrupted at the chord are called braces. The assembly of chord and braces is a joint.

The nodes of the space frame model shall be the intersection points (working points) of the centre lines of the braces with the centre line of the chord (see Figure 12.3-1). The offset is the distance between brace working points. Joints with zero offset are called *concentric*.

For a global analysis, offsets smaller than $D/4$ (see Figure 12.3-1) need not be included in the space frame model of the structure. However, for a local analysis, the axial forces in the brace members of K- and X-joints (see 14.2.4) cause secondary moments in the chord that can be important and should generally be assessed (see A.12.3.3.1).



Key

1 chord can

2 stub

3 brace

4 working points

D can diameter

c offset

d brace diameter

Figure 12.3-1 — Definition of offset

12.3.3.2 Member modelling

The structural model of a framework shall primarily consist of beam elements representing the axial, bending, shear and torsional stiffnesses of the structural members. In some cases special modelling arrangements are necessary for pile clusters and large diameter members provided for storage or flotation.

The structural members of the framework shall be modelled using one or more beam elements for each span between the nodes of the model of the framework. The number of beam elements depends on the element formulation, actions, and potential for local dynamic response. The element properties shall be varied along the spans to account for variations in the component cross-section where appropriate. The element properties shall also be adjusted to account for any corrosion allowance.

The structure shall be modelled in detail to include the primary and secondary structures, conductors, and appurtenances to ensure that action effects are accurately predicted. If this is not possible, the necessary detail of the model shall be prioritized as follows, in the order given.

a) The primary structure.

b) The secondary structure

   If representation of secondary framework is simplified, a more rigorous analysis of the secondary framework shall be undertaken for its local design with local analysis boundary conditions applied.

c) Components provided for temporary conditions such as launch framing, mudmats, etc.

   These shall be included in the model for appropriate analyses unless

   — they are removed prior to the design situation under consideration, or

   — the action effects on these components (and the effect of their stiffness on the rest of the structure) are not significant.

d) Conductors

   These shall be included in the model if they contribute significantly to the overall stiffness or strength of the structural system, including the foundation; otherwise, they may be treated as appurtenances (12.3.6).

e) Appurtenances.

Care shall be taken not to exclude secondary structure or appurtenances that can be subjected to excessive action effects induced by deformation of the primary structure.

Interaction with adjacent structures shall be considered and appropriate actions applied to represent any relative movement. Particular consideration shall be given to actions that do not result in overcoming friction at bearings, and actions resulting from pipe-work movements.

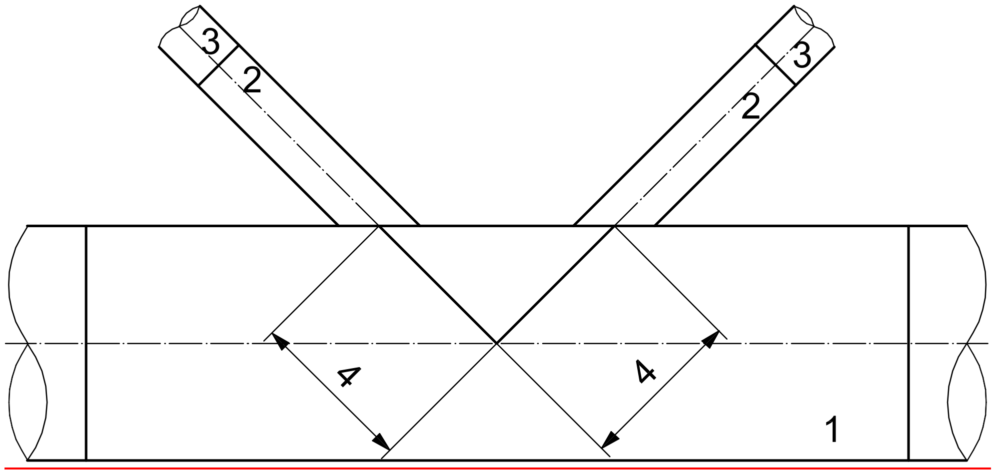
When the structural contribution of any component is neglected, the self weight, buoyancy and hydrodynamic actions on the component shall still be included in the model.

12.3.3.3 Joint modelling

For typical structures, depending on the diameter of the chord, the length between the physical end of the brace stub and the centre line of the chord can be significant, and can affect the calculation of member end forces and stresses, weights, masses, hydrodynamic and hydrostatic actions. In such cases, it is customary to model the length of braces between the outer surface of the chord and its centre line as rigid connections (rigid links); joint flexibility of brace and chord connections is thus neglected.

For joints between small diameter braces and large diameter chords, rigid links may be introduced between the longitudinal axis of the chord and the intersection of the chord can with the brace stub, see Figure 12.3.2.

Where rigid links are included, care shall be taken that they only contribute stiffness to the structural model; their stiffness shall be at least one order of magnitude greater than that of the brace and the brace stub, if present. Rigid links shall not contribute to the mass and shall not attract direct hydrodynamic or aerodynamic actions. Some computer analysis systems allow the introduction of member end offsets to achieve these requirements. Care should be taken when using elements with very high stiffnesses to represent rigid links, as computational problems can occur.



Key

1 chord can
2 brace stub
3 brace member
4 rigid link

Figure 12.3-2 — Definition of member rigid links

Joint flexibility is an elastic effect in which bending moments are redistributed. Where several braces connect with the chord at the same joint, the flexibility effects are dominated by the rotation of the brace ends as a result of chord wall rotation. This chord wall rotation results from the forces and moments in all the braces at the node and can increase or decrease any particular member end moments. In practice full joint modelling is rarely performed, where joint flexibility is modelled, it shall be modelled throughout the structure.

12.3.4 Material properties

Material properties (e.g. Young’s modulus, yield strength, ultimate tensile strength) used in the structural analyses of a new design shall reflect the materials specified for construction.

For existing structures, material properties may be based on statistical analysis of the material certificate data. Where no documentation can be found for assessing existing structures, coupon tests may be used to indicate which grades of steel were used to fabricate the structure.

Material properties used for non-linear analysis (e.g. true stress-strain curve) shall be based on published or test data for the appropriate steel specification and grades, including, if appropriate, strain hardening and plasticity effects (see also 12.6.7).

When assessing the resistance of a structure to elevated temperatures arising from fires, account shall be taken of the dependency of material properties, including yield strength, f_y, and Young's modulus, E, on temperature.

12.3.5 Topsides structure modelling

Where the structure and the topsides structure interact significantly, a combined model of structure and topsides structure shall be used. This applies equally to the design of new structures and the assessment of existing structures. The modelling of the topsides structure shall represent the interaction of the structures and the sequence of installation. The primary topsides structure shall be included in the structural model of the whole platform. The model shall include any deck members, deck plating and stiffening that affect or contribute to the distribution of forces into the structure. Differential deflections shall be taken into account when these can have a significant effect on the performance of the structures, particularly for serviceability conditions. The sequence of stressing introduced into rigid jointed structures arising from installation shall be considered.

If separate models for the structure and the topsides structure can be justified, the stiffness of the topsides structure and its interface with the structure shall be modelled in sufficient detail to ensure that its self weight and applied actions are appropriately distributed to the structure.

12.3.6 Appurtenances

The support and movement of appurtenances shall be modelled with appropriate constraint conditions so that the actions applied to them, including those due to self weight, are correctly transferred to the main structure. An example of an arrangement that often occurs is that unrestrained differential axial displacement and rotations between an appurtenance and its guide(s) are possible but that the guide(s) constrain the appurtenance to have the same lateral movements. The effect of any eccentricity in the support arrangement on the appurtenance and the structure shall be checked and included in the analysis if significant.

12.3.7 Soil-structure interaction

12.3.7.1 General

The stiffness of a piled foundation generally displays non-linear characteristics. The appropriate soil properties shall be determined in accordance with ISO 19901-4 and Clause 17.

The foundation shall be modelled and analysed using non-linear soil p-y, t-z and Q-z curves generated from site-specific data, when available. Particular care should be taken in modelling relatively thin layers near the sea floor, where p–y soil properties change rapidly. The soil layers shall be modelled accurately to at least the depth at which the piles are expected to terminate. If the pile tip rests on weak soil layers, the possibility of punch-through shall be investigated.

For an analysis to determine the behaviour of the structure, an equivalent simple linear elastic model may be used to model a piled foundation. When such a simple model is used, the forces and displacements at the pile heads (at the sea floor) shall be checked for compatibility with the predicted non-linear foundation behaviour.

Pile penetrations shall be conservatively assessed, based on upper and lower bound considerations of soil properties and pile driving hammer properties. Either upper or lower bounds can be more critical for different situations and different components of the overall structural system.

The effects of global sea floor scour, of local scour in granular soils and of the partial loss of soil-pile contact in cohesive soils shall be taken into account.

12.3.7.2 Pile groups

When modelling the individual piles in a pile group (or cluster), the non-linear soil p-y, t-z and Q-z curves shall be adjusted to account for pile group effects. Alternatively, an interaction model to account for the interaction between individual piles shall be included in the analysis. See also 17.9.

For pile groups, each pile-sleeve connection may be represented by one or more coaxial tubular member(s). Any bracing, shear plates or similar connections between a pile-sleeve and a leg shall be simulated by an appropriate combination of bar, beam and plate elements that models the method of transfer of forces between the leg and pile-sleeve connection.

12.3.7.3 Pile connectivity

The constrained movement of ungrouted piles within legs shall be modelled. Each pile shall be allowed to move independently of its leg, both along its axis and around all rotational axes, but the leg and piles shall be constrained to have the same lateral displacements (normal to the longitudinal axis of the leg/pile). Such constraints should be applied at locations where contact is expected to occur, such as at shims or centralizer locations or at plan bracing levels.

12.3.7.4 Conductor modelling

Conductors can contribute significantly to the foundation stiffness and lateral strength of a structure. In such cases, the conductors shall be modelled and analysed as structural components with their own foundation stiffness.

The lateral behaviour of the conductors below the sea floor may be modelled in the same way as piles, either as equivalent members or individually, employing non-linear soil p-y curves generated from site-specific data when available. For vertical, self-supporting conductors that are constrained only in the horizontal direction, modelling the conductors' axial behaviour using the soil t-z and Q-z characteristics may be omitted. Group effects of conductors shall be considered.

When equivalent members are used, the displacements at the sea floor shall be checked for compatibility with the predicted non-linear foundation behaviour.

It should be noted that the installation of conductors can cause significant disturbance to the integrity of the soil and the contribution from the conductors to the foundation stiffness can be much lower than anticipated. Where the foundation stiffness associated with the conductors is included in the analysis, consideration shall be given to the effectiveness of the conductor foundations and to situations where not all of the conductors are installed. In the latter cases, although the hydrodynamic actions on the structure are lower, the action effects on the main structure piles can be higher. An upper bound value of the action effects on the main structure piles should be determined by releasing the horizontal and bending (but not torsion) restraints at the base of each conductor. An upper bound value of the action effects on any conductor bracings close to the sea floor should be determined by modelling an upper bound foundation stiffness.

12.3.7.5 Conductor connectivity

The constrained movement of conductors within their guide frames shall be modelled. Each conductor shall be allowed to move independently of its guide, both along its axis and around all rotational axes, but the conductor and its guide shall be constrained to have the same lateral displacements (normal to the longitudinal axis of the conductor) as the guide. The contact action, including friction, between curved or slanting conductors and their guide frames requires special consideration.

12.3.8 Other support conditions

The action of water pressure on a structure (or on a supporting barge) while floating (e.g. during transportation or installation) shall be evaluated from suitable hydrostatic and hydrodynamic analyses for wave and current, as appropriate, and shall be applied to appropriate external surfaces of the structure (and/or the supporting barge). See also 12.4.3.2

Where balanced actions are applied to a structure or component, the minimum number of artificial restraints shall be applied for certain types of analysis (e.g. lift) to prevent rigid body movement. Artificial restraints shall be applied as follows:

— as translational restraints that do not inhibit relative deflections within the structure or the component;

— at points where no direct actions are applied, although distributed self weight may be applied at a restraint;

— to extreme locations in the structure or the component; and

— to relatively stiff points in the structure or the component.

Where components of the structure are not fully restrained, such as conductors within guides (see 12.3.7.5) and bearing surfaces between topsides structure and bridge structure, consideration shall be given in the analysis for movement at such interaction points.

12.3.9 Local analysis structural models

Local models may be created to study the behaviour of a structural component in greater detail. The boundaries of the local model shall be sufficiently remote from the component being studied that the method of applying the boundary conditions does not affect the component's behaviour. The actions applied to the local model shall be the same as those applied to that part of the global model represented by the local model; the application of the actions to the local model may be more detailed than in the global model.

The boundary conditions applied to the local model shall be one of the following:

— forces extracted from an analysis of the overall structural system and applied as direct actions at the boundaries of the local model together with the minimum number of restraints to prevent rigid body movement;

— displacements extracted from an analysis of the overall structural system and applied as imposed deformations at the boundaries of the local model;

— a set of related forces, displacements and stiffnesses determined by substructuring the overall structural system in which one or more substructured parts represent local models with connections to the remainder of the structural system at the intended boundaries of the local model.

In the absence of forces, displacements and stiffnesses at the boundaries of the local model derived from the overall structural system, appropriate conditions shall be applied to the boundary of the component to simulate the behaviour of the surrounding structure. Where there is uncertainty over the boundary conditions, a range of possible conditions shall be considered.

12.3.10 Actions

In a structural analysis, the applied actions shall include partial action factors appropriate to the action being considered and the limit state checks to be performed; it is common practice to subdivide actions into categories to allow separate application of partial action factors and to provide flexibility in the analysis.

Actions shall be determined by recognized methods, taking into account the variation of actions in time and space, relevant environmental and soil conditions and the limit states being addressed, see Clauses 8 to 11.

Permanent and variable actions shall be based on the most likely representative values for the situation being analysed. Consideration shall be given to both representative maximum and minimum values. In a number of cases representative minimum values can govern the design of some components.

Hydrostatic pressures shall be based on the likely range of fluid surface elevations and densities. Hydrostatic pressures on structures while afloat, such as during transportation, installation and removal, shall include the effects of tilt of the structure due to wind, wave action or damage.

The action effects due to wave, current and wind actions shall include the influence of such actions on structure motions when the structure is afloat or on a transportation barge. In cases where dynamic action can be of importance, such influence shall be considered in determining action effects. Quasi-static or dynamic analyses shall be used in accordance with 12.5.

Less well defined actions, such as variations in topsides weight and centre of gravity and actions during loadout, should be represented by ranges of representative values, the structure being checked for the most onerous values.

Actions on a local model shall be included in that model (see 12.3.9).

12.3.11 Mass simulation

For dynamic analysis and for motion prediction during transportation and installation, a suitable representation of the mass of a structure is required. Generally, masses should be determined as a representative combination of mass values for the situation considered. Such a mass simulation shall include relevant contributions from, at least, the following:

- all structural components and appurtenances;
- the operating mass of all intended equipment;
- the estimated mass of temporary items, such as slings, storage, lay-down, etc.;
- masses of any fluids contained within the structure or topsides, including vessels, piping contents, oil and bulk storage, etc.;
- snow and ice accumulation on the structure, if significant;
- drill cuttings or other deposits on the structure, if significant;
- the mass of marine growth;
- the added mass representing hydrodynamic reaction to structure motions (allowing for any marine growth thickness);
- the internal mass (flooded members) moving with the structure.

The magnitude and distribution of mass shall be as accurate as necessary for the determination of all significant rigid body motions or modes of vibration for the structural analysis being performed. Particular attention shall be paid to the location of individual centres of mass, in particular for heavy equipment and storage tanks in the topsides structure. Sufficient dynamic analyses shall be carried out to determine the sensitivity of action effects to anticipated variations in the magnitude and distribution of mass.

Calculation of the hydrodynamic reactions to structure motions (represented in the form of added mass and damping) shall be based on the best available published information or suitable hydrodynamic analysis. In lieu of such analysis, the added mass may conservatively be taken as equal to the full mass of water displaced by the structure.

12.3.12 Damping

Damping arises from a number of sources, including material damping, soil damping, hydrodynamic damping and frictional damping between moving parts. For fixed steel offshore structures, damping is relatively small. In the absence of substantiating values obtained from measurements of existing structures conservative (lower bound) damping values shall be used.

12.4 Analysis requirements

12.4.1 General

All structural analyses performed shall simulate, with sufficient accuracy, the action effects in the structural system or the structural component for the limit state being considered. This shall be achieved by appropriate selection of the analysis type and method of solution with due regard to the nature of the actions and the nature of the structural behaviour. More than one analysis type can be required to simulate the behaviour of the structural system during a given phase of its life.

Sufficient global analyses of the structural system shall be performed to allow subsequent assessment of the structural components. As a minimum, the structural system shall be assessed for ultimate limit state (ULS) conditions. Local analysis shall be performed to assess any detail that is complex in form and/or appears from the global analysis to experience large action effects. Such local analysis may be based on non-linear methods.

Table 12.4-1 summarizes the applicability of analysis types for different design situations.

Table 12.4-1 — Applicability of analysis types for design situations

Situation	Analysis type			
	Static/Quasi- static linear elastic	Dynamic linear	Non-linear	Structural reliability
1. Pre-service and removal situations				
1.1 Fabrication	Appropriate			
Fatigue during fabrication		Appropriate (needed for wind induced vibrations for long slender braces only, see Clause 16)		
1.2 Loadout	Appropriate			
1.3 Transportation	Appropriate			
Fatigue during transportation		Appropriate (needed for long sea tows only, see Clause 16)		
1.4 Installation	Appropriate	Appropriate for fatigue analysis of piles during pile driving		
1.5 Removal	Appropriate		Appropriate if needed to demonstrate that no failures will occur during removal	
2. In-place situations				
2.1 Normal operating and extreme environmental situations	Appropriate for structures that are not unduly dynamically sensitive, see 9.9	Appropriate for structures that are appreciably dynamically sensitive	Appropriate for foundation components	Appropriate, but outside the scope of this International Standard
Fatigue during the in-place situation	Appropriate for structures that are not unduly dynamically sensitive, see Clause 16	Appropriate for structures that are appreciably dynamically sensitive, see Clause 16		
2.2 Accidental situations	Appropriate, but can be unduly conservative	Appropriate, but can be conservative	Appropriate	Appropriate, but outside the scope of this International Standard
2.3 Seismic events	Appropriate for extreme level earthquake (ELE) event, see Clause 11	Appropriate (see Clause 11)	Appropriate for abnormal level earthquake (ALE) event, see Clause 11	Appropriate, but outside the scope of this International Standard

NOTE 1 Quasi-static linear analysis includes inertia actions to represent rigid body structural accelerations.

NOTE 2 Linear static or quasi-static analysis as defined here include simple non-linearities, e.g. from gaps opening and closing.

NOTE 3 Non-linear local analysis for component buckling shall be included (e.g. using the provisions of Clause 13).

NOTE 4 Non-linear global analysis may be used to demonstrate that a structure is safe when a linear analysis predicts failure. The assumptions used in the non-linear model shall be demonstrated to be pessimistic.

NOTE 5 All the analyses in the table shall be performed using partial action and resistance factors as specified in Clauses 9 to 11 and 13 to 15, respectively. In cases where these partial factors are deemed inappropriate, the analyses may be performed within a structural reliability framework. When reliability methods are used, the input data and methods used shall be demonstrated to be satisfactory.

12.4.2 Fabrication

Fixed steel offshore structures, particularly large or slender structures, or those with particularly slender structural components, should be reviewed to determine whether analysis for fabrication is required.

Where such analysis is undertaken, consideration shall be given to the sequence and to the completeness of fabrication (e.g. whether welding has been completed at particular joints) in determining action effects. Specific consideration shall also be given to the stability and strength of structural components during fabrication and to the need for any temporary supports or strengthening. Adequate support for equipment subjected to temporary actions, such as for crane footings, shall be demonstrated.

Fabrication phase analyses may generally be performed using static analysis. However, dynamic analysis shall be considered for the determination of action effects in tall, slender structures and long thin appurtenances in view of vibrations induced by wind turbulence.

12.4.3 Other pre-service and removal situations

12.4.3.1 General

Loadout, transportation and installation (including launch, floating, upending and/or lifting) are situations in which the structure is in motion and mass inertial forces due to accelerations need to be considered. In most cases, in particular for loadout and installation, static linear elastic analysis is adequate, but care shall be taken in the definitions of the actions and the application of partial action factors to ensure that only feasible and stable conditions are analysed. In general, the analysis should be undertaken for unfactored actions, with action effects factored after the internal forces are calculated (see 8.2.4.2).

Requirements for the calculation of actions and partial action factors for pre-service and removal situations are given in Clause 8.

12.4.3.2 Loadout

Loadout refers to the series of operations needed to transfer the structure from its assembly position in the fabrication yard to the transportation or launch barge. During loadout, the structure is generally supported unevenly, with changing support conditions. Limitations on the acceptable support conditions shall be identified during the design phase, and shall not be exceeded during loadout. Because loadout occurs slowly, inertial actions may be neglected, and linear static analysis is sufficient. However, the non-linearities from lift-off of supports should be taken into account.

12.4.3.3 Transportation

Fixed steel offshore structures and topsides are normally transported to location by barge. During transportation, the structures are subjected to actions generated by heave, sway, surge, pitch and roll accelerations. Parts of the structure overhanging the barge are also susceptible to actions due to buoyancy, wave action and wave slam if the structure penetrates the water. Structural components can also be subjected to excitations associated with wind induced vortex shedding. These action effects should neither exceed the strength of structural components nor cause excessive fatigue damage.

The structural integrity of a steel structure shall be assessed for the transportation phase (sea tow) of the structure, whether it is self-floating, barge-supported or barge-assisted. Analysis of situations during sea tow may normally be based on quasi-static analysis, representing the motion of the steel structure due to the most onerous combination of heave, sway, surge, pitch and roll accelerations, as predicted by hydrodynamic analysis. For a long sea tow, where the duration of the tow precludes assurance of calm weather for the whole length of the tow, fatigue damage shall be assessed (see Clause 16).

The model of the structure (see 12.3) shall include all sea-fastenings. If the bending and torsional stiffness of the barge is significantly greater than that of the structure, the barge may be assumed to be rigid and appropriate boundary conditions applied where the sea-fastenings are connected to the deck of the barge. A quasi-static analysis shall be carried out for design combinations of self weight, roll, pitch and heave, with roll and pitch accelerations applied about the appropriate roll and pitch centres for the combined barge and structure, with the barge and structure inclined at the maximum angle of roll and/or pitch.

If barge flexibility is important, the model shall include the structure and the barge, as well as a representation of the hydrostatic and hydrodynamic actions due to the surrounding water. The barge should preferably be modelled using plate/shell elements, having both in-plane (membrane) and out-of-plane bending stiffness. The total actions on the barge and the structure, with the water pressure distribution derived from a motion analysis, should be in quasi-static equilibrium.

Sea-fastening and grillage supporting the structure on the barge shall be checked for strength using design situations including appropriate combinations of

— self weight,

— roll and pitch angles, and

— roll, pitch and heave accelerations.

The modelling shall recognize the sequence of loadout of the structure onto the barge and the attachment of the sea-fastenings. In particular, as sea-fastenings are installed after the structure loadout on the transport barge is completed, it is common practice to assume that they do not resist any of the structure dead weight in the still water condition. The validity of this assumption should be verified in consideration of the relative flexibility of the barge and the structure.

Structural components, sea-fastenings and grillage shall be checked for strength in accordance with Clauses 13 to 15. The fatigue life of structural components and sea-fastenings shall be assessed, when relevant, in accordance with Clause 16. Structural components that are subjected to wave slam shall be assessed for strength and fatigue due to the associated actions.

12.4.3.4 Installation

Structures are either launched or lifted into the water. When launched, a time-history analysis shall be performed

a) to determine the launch trajectory to ensure sufficient clearance from the sea floor,

b) to determine the stability of the structure after separating from the barge,

c) to check that the structure is not overstressed as it

   1) rotates and enters the water,

   2) upends, with or without crane assistance, and

   3) is ballasted so that it rests on the sea floor, and

d) to check that the barge is not overstressed at any stage of the launch.

The strength of the structure shall be checked using representative actions generated at various stages of the launch and upending process. The actions shall include hydrostatic and hydrodynamic actions on the structure, actions from self weight at the appropriate orientation relative to the structure and the actions due to the instantaneous translational and rotational accelerations of the structure. Sufficient boundary conditions shall be applied to prevent rigid body movement of the structure and to provide a datum for deflections. The structure shall be analysed for each set of representative actions.

Transients are important for lift installations, but a full dynamic analysis is not normally carried out. Instead static analysis is performed employing representative actions to cover motions of the structure and crane vessel, actions due to "snatching" as the structure is lifted off the barge and due to impact between the structure and the sea floor, etc. As with the launch, sufficient boundary conditions shall be applied to prevent rigid body movement and to provide a datum for deflections. Except at the lifting points, the support reactions should be negligible. When performing single-point lift analysis, it is often convenient to determine the centre of gravity and orient the model so that the centre of gravity is vertically below the lifting point. The effects of variances in the lengths of the slings should be taken into account.

Installation of the topsides by lifting does not normally impose actions on the structure that are greater than, or in a different direction to, those induced by the topsides in in-place situations. Accordingly, analysis of the actions on the structure due to the installation of the topsides may be omitted.

For float-over topsides installation, the design of the structure legs and other components can be substantially affected by the lateral actions induced by the topsides/barge system. This design situation shall be appropriately analysed.

For each installation analysis, structural components shall be checked for strength in accordance with Clauses 13 to 15. In particular, structural components and appurtenances attached to the structure can be subjected to significant slamming action upon entering the water. Consideration shall be given to the stresses induced by slamming.

The structure shall also be assessed for on-bottom stability, when the structure is resting on the sea floor with the aid of mudmats prior to piling being completed; the cases considered shall include that of the structure with and without pile(s) stabbed.

12.4.3.5 Removal

Analysis of the structure for removal shall represent the structure in all appropriate configurations, and shall include the suction and pull out resistances of pile stubs (i.e. any length of pile attached to the structure following pile cutting) prior to separation from the sea floor. Weights of accumulated debris and marine growth left in place at the time of removal shall also be considered, together with the weights of the piles and pile grout to be lifted during removal.

The effect of any structural damage, including degradation of the materials during the life of the structure shall be considered. The determination of the material degradation shall take into account any results from underwater surveys and inspections.

12.4.4 In-place situations

12.4.4.1 General

The design of a fixed steel structure for in-place situations can generally be based on a static or quasi-static analysis, using a linear elastic model of the structure coupled with a non-linear foundation model, unless there is a likelihood of significant dynamic or non-linear response to a given type of action. In such cases, linear dynamic or non-linear static or dynamic analysis approaches, respectively, are normally required, as described in 12.5.

It is generally acceptable to base static structural analysis on the principles of deterministic analysis, predicting action effects to specific events.

The structure shall be analysed for operating and extreme environmental actions. Clause 9 defines the partial action factors for permanent and variable actions for in-place situations.

Selection of environmental parameters for the analysis of in-place situation(s) shall be in accordance with ISO 19901-1. The analysis shall ensure that sufficient variations and combinations of the parameters are analysed to capture the most onerous action effects throughout the structure.

If soils data are available, a pile-soil interaction analysis shall be carried out using non-linear soil p-y, t-z and Q-z curves (see 12.3.7). When, during preliminary analysis, soils data are not yet available, a simplified model of the foundation may be used. Such a simplified model may also be used if design or assessment of the foundation and the lower portion of the structure is not the purpose of the analysis.

The foundation capacity shall be verified in accordance with Clause 17 and the strength of structural components in accordance with Clauses 13 to 15.

12.4.4.2 Extreme environmental conditions

To determine the governing action effects for each structural component, the structure shall be analysed for environmental actions due to the most onerous combinations of wind, current and wave conditions, as specified in Clause 9. To achieve this, a sufficient number of appropriate combinations of the following environmental parameters shall be considered:

- water depth, including effects of tides and storm surge;
- wind speed, omnidirectional or in relation to wind direction;
- current speed, omnidirectional or in relation to current direction;
- wave height and period, omnidirectional or in relation to wave direction;
- wave position (phase angle) relative to the structure.

In regions with seasonal ice cover, actions due to ice also shall be considered.

12.4.4.3 Accidental situations

Analyses for accidental situations shall be performed in accordance with Clause 10.

The analysis of collisions (such as ship, helicopter or iceberg collisions) may be performed by considering the energy absorption arising from the combined effect of local and global deformation. The energy absorbed shall be compared with, and equated to, the impact (kinetic) energy due to collision, and the results shall be documented.

The analyses should allow for

- the energy absorbed in local deformation of the structures at the point of impact, e.g. denting of the colliding ship and denting of a structural component or a group of structural components, and
- the energy absorbed in elastic and plastic deformation of the structural system, e.g. using quasi-static non-linear analysis of the structural system under the estimated impact.

The resistance of the impact area may be studied using local models. These will generally involve non-linear geometrical and material analyses, since the local structure may be allowed to yield and deform substantially under the accidental actions. Appropriate boundary conditions should be applied far enough away from the damaged region for inaccuracies to be minimized.

Global analysis of the structure under accidental situations, including those due to dropped objects, fire and explosion, shall normally be required to ensure that a progressive collapse is not initiated. The analysis shall include the weakening effect of damage to the structural system in the affected area. Linear elastic redundancy analysis may be used to demonstrate that removal of a damaged member will not initiate progressive collapse. Non-linear analysis may be used to simulate the redistribution of action effects as section resistances are exceeded. The global analysis may be based on a simplified representation of the structure that is sufficient to simulate progressive collapse.

The natural frequencies of the structural system shall be estimated and compared with the frequency content of the accidental actions. Analysis of the overall structural system shall include any significant dynamic response under these actions.

12.4.4.4 Seismic events

Analyses for seismic events shall be performed in accordance with Clause 11 for the extreme level earthquake (ELE) and the abnormal level earthquake (ALE). The structure shall sustain little or no damage due to ELE events for which linear analysis is appropriate. During ALE events, the structure may sustain considerable local damage (but not a total collapse), provided structural failures do not cause loss of life and/or environmental damage. Non-linear analysis can be required to demonstrate conformance with these safety requirements.

12.4.4.5 Fatigue analysis

Requirements for, and guidance on, performing a fatigue analysis are provided in Clause 16.

12.4.4.6 Analysis for reserve strength

Reserve strength analyses can be used to determine the adequacy of unconventionally framed structures and for the assessment of existing structures (see 7.10). They are intended to identify the collapse strength of the space frame structure and foundation under the action of abnormal environmental actions. The uncertainties associated with foundation capacity are significantly greater than those associated with the ultimate strength of space frame structures, see A.12.5.4. In performing reserve strength analyses, it is therefore important to make this distinction and to evaluate both structural and foundation failure modes. Owing to this, the following strategy is recommended:

a) structural or foundation failure should be identified using an analysis based on the mean (or best estimates) of structural steel properties and soil properties;

b) where foundation failure occurs before structural failure, structural failure should be determined by assuming a foundation capacity based on upper bound estimates of soil properties. The upper bound approach, b) above, provides an assessment of the steel structure strength.

In each case, the foundation should be included in the model in order to evaluate potential combined structure and foundation collapse mechanisms.

For dynamically responding structures (see 12.5.2), dynamic effects can influence collapse due to environmental actions.

Reserve strength evaluation is used to estimate the most likely collapse strength of a structure with partial resistance factors set to 1,0. Since partial resistance factors are omitted, an ultimate strength evaluation shall be interpreted and used with care.

12.5 Types of analysis

12.5.1 Natural frequency analysis

A natural frequency analysis shall be carried out to determine whether dynamic behaviour is significant. A reasonably accurate structural model, including both stiffness and mass, shall be developed and analysed so that the natural frequencies of the structural system can be compared to the frequencies of any excitations.

Dynamic behaviour is likely to be significant if any natural frequency, particularly the fundamental frequency, is similar to the frequency of an excitation. Analyses should be carried out to determine any sensitivity to structural stiffness and mass, and to foundation stiffness.

12.5.2 Dynamically responding structures

Structures for which dynamic behaviour is significant are generally referred to as dynamically responding structures. Redundant, multi-legged fixed structures (e.g. jackets, towers, etc.), with fundamental natural periods or having one or more components with natural periods greater than 2,5 s to 3 s usually respond dynamically to wave action during sea tow or in-place situations. For other types of structures, such as monotowers and caissons, dynamic behaviour can be significant even with natural periods of 1 s or less.

Other actions to which the structural system can be subjected, such as wind turbulence, seismic events, impact and explosion, can also cause dynamic effects of significant magnitude at other natural periods.

12.5.3 Static and quasi-static linear analysis

Static analysis is appropriate when dynamic effects are minimal and can be assumed to be covered by the partial action and resistance factors.

Quasi-static analysis is appropriate when dynamic effects can be assumed to be approximately uniform throughout the structural systems and so small that one static analysis or a series of static analyses, with a small correction for dynamic effects, is adequate. The correction for dynamic effects may be determined using one or the other of the following methods:

a) by performing one static analysis in which the actions are enhanced by a set of equivalent quasi-static inertial actions representing the dynamic response;

b) by performing a series of static analyses over an appropriate range of frequencies of excitation, where, at each frequency, the corresponding actions are applied and the calculated action effects are multiplied by the dynamic amplification factor (DAF) of a single degree of freedom (SDOF) system at that frequency.

Guidance on the application of the chosen method is given in A.12.5.3.

12.5.4 Static ultimate strength analysis

The collapse strength of a space frame structure is usually expressed as the ratio of the global environmental action at collapse, measured in terms of base shear or overturning moment, to the global environmental action used for design; see 7.10. The collapse strength of the structural system may therefore be defined by the maximum value of the global environmental action sustained by the structural system during the analysis; guidance for the analysis is provided in A.12.5.4.

The following analysis procedure is suggested. Unfactored permanent and variable actions are applied followed by the environmental actions due to wind, wave and current. The environmental action is gradually increased until structural collapse occurs.

For a static strength analysis, the following checks shall be performed:

a) assess the deformation patterns of the structural system to identify whether a clear failure mechanism has developed;

b) check the plastic strains in all members that have yielded in tension against the ductility limits set by fracture considerations (see 12.6.6);

c) check the internal axial forces acting on the pile heads against pile capacities and performances;

d) check the joint forces against the joint strength.

The structural behaviour should also be studied to determine how forces are redistributed as members, joints and piles fail.

12.5.5 Dynamic linear analysis

When dynamic response is considered significant, the structural system shall be analysed for dynamic behaviour. An accurate structural model including both stiffness and mass shall be used. Lumped and consistent mass modelling shall be employed as appropriate.

The type of analysis is governed by the form of applied actions:

— steady state analysis in response to harmonic actions, as required for spectral analysis;

— transient analysis in response to arbitrary time-history actions, as can be required for accidental situations and non-linear actions due to waves or earthquakes.

For both types of analysis, the behaviour of the structure and the foundation are assumed to be linear elastic.

12.5.6 Dynamic ultimate strength analysis

Dynamic non-linear structural analysis may be performed in one or the other of the following ways.

— Full transient dynamic non-linear analysis, in which the environmental action is simulated in time.

— Quasi-static analysis, in which static non-linear analysis procedures are used, with the environmental actions enhanced by a set of equivalent quasi-static inertial actions representing the dynamic response. The set of equivalent quasi-static inertial actions may be determined in a manner analogous to the procedures in 9.8 and A.9.8. A static pushover analysis can then be performed using the procedure suggested in 12.5.4.

12.6 Non-linear analysis

12.6.1 General

When, for example, the structure is subjected to abnormal environmental actions due to wind, wave and current or an earthquake, or to accidental actions from ship impact, fire or explosion, and when a linear analysis predicts

— displacements of a magnitude that are likely to cause second order P–Δ effects,

— joint failure,

— member buckling, and/or

— stresses that exceed the yield strength of the material,

then non-linear analysis may be performed to demonstrate that overall structural integrity is not impaired.

Non-linear analysis shall include all relevant actions to represent the maximum actions to which the structure or component will be subjected. The actions shall be applied to simulate the action history, e.g. the actions due to permanent and variable actions shall be applied first, to be followed by the actions due to wave and current. The modelling of action history is necessary because non-linear behaviour is dependent on the sequence of the force distribution through the structure. The magnitude of the actions should be the representative values, i.e. without partial action factors. Similarly, no partial resistance factors should be applied to the resistances. Any suspected sensitivity of behaviour to material properties, e.g. Young's modulus and yield strength, shall be investigated by additional non-linear analyses with modified material properties (see 12.6.7).

The behaviour of the structural system to the applied actions can be static or dynamic (see 12.5.2). Depending on this a (quasi-)static or transient dynamic non-linear analysis shall be performed in accordance with 12.5.4 or 12.5.6.

As a non-linear analysis is undertaken without partial factors, an appropriate margin of safety — dependent on the situation being considered — should be included in the interpretation of the results.

12.6.2 Geometry modelling

In addition to the requirements of 12.3, the following apply.

When creating models for a non-linear analysis, due consideration shall be given to the types of elements and size of mesh required to predict non-linear structural behaviour. For example, sufficient elements shall be employed along the length of a member to simulate elasto-plastic buckling involving yielding at the ends and middle of the member; insufficient elements will overestimate the strength of the member. In addition, sufficient elements shall be employed to simulate all possible modes of failure — especially beams that may fail due to lateral torsional buckling. Thus, a model for non-linear analysis will usually be more complex than the model required for extreme environmental and fatigue analysis.

12.6.3 Component strength

The collapse of a space frame structure usually results from progressive failure of its components, in particular its primary members and/or joints. Correct modelling of component behaviour is therefore essential for predicting non-linear behaviour. This may be achieved using a finite element formulation or phenomenological models.

The redistribution of internal forces following a component failure, and the prediction of collapse behaviour, therefore depends on the accuracy of the models used to describe the full path of member and joint behaviour, from their initial stress-free state to failure and including their post-failure response.

12.6.4 Models for member strength

The ultimate strengths of undamaged tubular members may be established using the formulae given in Clause 13, with all partial resistance factors set to 1,0. The models for describing the ultimate strength and the subsequent post-collapse behaviour of members in a space frame structure shall be able to describe the following failure modes:

— tensile and compressive material yielding of a member's cross-section, which may be achieved by adopting a plastic hinge formulation or by monitoring the stresses and strains in the member;

— buckling of a member and the post-buckling redistribution of internal forces that can involve local buckling; for open section members this includes both Euler and lateral torsional buckling as well as taking the interaction between yielding and buckling into account;

— local buckling.

The models should allow for the effects of initial geometric imperfections and hydrostatic pressure. Consideration should be given to any locked-in forces due to fabrication.

12.6.5 Models for joint strength

The ultimate strengths of undamaged tubular joints may be established using the formulae given in Clause 14, with all partial resistance factors set to 1,0. If both the ultimate strengths for axial forces and moments, as provided by these joint strength formulae, exceed the axial yield force and plastic moment strengths of the brace cross-section, the brace will fail before the joint. If this is the case, joint failure does not need to be included in the structural model. If it is not the case, joints which can participate in the failure mechanism shall be identified and their behaviour included in the non-linear structural model.

12.6.6 Ductility limits

When joints are stronger than the incoming brace members, the model for member tensile failure shall include a ductility criterion that will disconnect (sever) a member end when the plastic strains have reached the tensile fracture strain. In lieu of more advanced analysis, the member end shall be conservatively assumed to disconnect when the tensile strains in the extreme fibres of a steel member exceed 5 %.

12.6.7 Yield strength of structural steel

If the mean yield strength of the steel for a part can reliably be determined, this mean yield strength may be used for the part under consideration instead of the specified minimum yield strength (SMYS) of the material. The SMYS is the representative value of the yield strength and should be used in all other cases.

The mean yield strength can be determined from either generic studies of the specific grades or from statistical analysis of coupon tests from particular plates used in the specific structure. However, coupon tests from the particular material used in the structure may only be used if they belong to a single population, see A.12.6.7. Benefit may be taken of strain rate effects; however, the strain rates used for nominally static tests shall be taken into account.

Increased strength due to strain hardening shall be considered carefully, as often there are uncertainties in the post-yield stress-strain behaviour:

- the effect is important in reducing strains in plastic hinges;
- for compression members, strain hardening only occurs in the post-collapse range and reduces the amount of internal force redistribution;
- for tensile members, ductility criteria limit strains to values which have to be exceeded for strain hardening to become significant;
- for joints, strain hardening will, to some extent, be included in the ultimate strength formulae.

12.6.8 Models for foundation strength

The foundation shall be included in the overall model and shall be analysed as a fully integrated part of the structural system using non-linear soil p-y, t–z and Q-z curves. For detailed design, these curves shall be generated from site-specific data. The effect of cyclic actions on the soil curves shall be considered. The model shall include the non-linear geometric and material behaviour of the piles.

Simplified models for foundation failure may be used when modelling collapse mechanisms which involve part of the framework and when structural failure occurs before foundation failure.

For non-linear ultimate strength analysis of the structural system, it can be more appropriate to use mean soil properties as opposed to representative properties. Representative soil properties shall be used to determine a lower bound of the system collapse strength if system collapse is governed by the foundation.

12.6.9 Investigating non-linear behaviour

Having performed a non-linear analysis of a structural system or one of its components, the magnitude of the stresses, particularly the equivalent von Mises stresses, shall be examined. The plastic strains shall also be examined to ensure that the ductility limit (fracture strain) has not been exceeded (see 12.6.6), and to confirm areas of yield. For analyses involving high temperatures, the material properties are significantly less than those at a normal ambient temperature of 15 ºC.

The deformation of members provides information on their behaviour, e.g. members failing due to elastic buckling will have large deformations but no areas of yield, while members yielding in tension will be straight and have areas of yield that can extend along the whole length of the member.

Any excessive rotations at the ends of members shall be examined to determine whether they are due to yielding of the member (plastic hinge) or to joint failure. The response of failed joints shall be checked to ensure that the forces in the chord and braces are consistent with respect to the defined behaviour, see 12.6.4 and 12.6.5.

The structural deflections shall also be checked to ensure that serviceability requirements are satisfied with regard to appurtenances such as risers and caissons, failure of which can necessitate platform shutdown or impair the platform’s safety systems.

The behaviour of the structure in a sequence of high waves shall be considered. If repeated plastic strains occur then a low cycle fatigue analysis shall be carried out.

The above checks shall be carried out for various stages of the non-linear analysis in order to gain an understanding of the history of the structure's behaviour, in particular the sequence of component failures and internal force redistribution.

13 Strength of tubular members

13.1 General

The requirements given in this clause apply to unstiffened and ring stiffened cylindrical tubulars having a thickness $t \geqslant 6$ mm, a diameter to thickness ratio $D/t \leqslant 120$ and material meeting the general requirements of Clause 19. In addition, yield strengths shall be less than 500 MPa and the ratio of yield strength as used to ultimate tensile strength shall not exceed 0,90.

The requirements for the different components and types and combinations of actions are contained in different subclauses, as detailed in Table 13.1-1. Where no reference is included for a particular component and actions, there exists insufficient test data to enable comprehensive design equations to be prepared, and these circumstances shall be assessed on a case-by-case basis.

Table 13.1-1 — Arrangement of requirements for members

Subclause	Component	Actions				
		Tension	Compression	Bending	Shear	Hydrostatic
13.2.2	Tubular	X				
13.2.3	Tubular		X			
13.2.4	Tubular			X		
13.2.5	Tubular				X	
13.2.6	Tubular					X
13.3.2	Tubular	X		X		
13.3.3	Tubular		X	X		
13.4.2	Tubular	X		X		X
13.4.3	Tubular		X	X		X
13.6.3	Cone	X	X	X		
13.6.4	Cone	X	X	X		X
13.7.2.2	Dented tubular	X				
13.7.2.3	Dented tubular		X			
13.7.2.4	Dented tubular			X		
13.7.2.5	Dented tubular				X	
13.7.3.1	Dented tubular	X		X		
13.7.3.2	Dented tubular		X	X		
13.8	Corroded tubular	X	X	X	X	X
13.9.2.2	Grouted tubular	X				
13.9.2.3	Grouted tubular		X			
13.9.2.4	Grouted tubular			X		
13.9.3.1	Grouted tubular	X		X		
13.9.3.2	Grouted tubular		X	X		

For tubulars subjected to hydrostatic pressure, it can be necessary to ensure that the circumferential value of yield strength is consistent with the value adopted in design.

The requirements in this clause assume that the tubular is constructed in accordance with the fabrication tolerances given in Clause 21. The requirements allow structural design to proceed on the basis that stresses due to the forces from the capped-end actions of hydrostatic pressure are either included in or excluded from the analysis.

In Clause 13, y and z are the axes of a tubular cross-section used to define in-plane and out-of-plane behaviour respectively. “In-plane” is the plane common to the longitudinal axis of the brace member under consideration and the longitudinal axes of the chord member providing restraint, while “out-of-plane” is perpendicular to this plane.

In Equations (13.2-1) to (13.9-39), the stresses are always the absolute values of the stresses as computed, and hence — whether tensile or compressive — are always positive.

13.2 Tubular members subjected to tension, compression, bending, shear or hydrostatic pressure

13.2.1 General

Tubular members subjected independently to axial tension, axial compression, bending, shear, or hydrostatic pressure shall be designed to satisfy the strength and stability requirements specified in 13.2.2 to 13.2.6.

13.2.2 Axial tension

Tubular members subjected to axial tensile forces shall be designed to satisfy the following condition:

$$\sigma_t \leqslant \frac{f_t}{\gamma_{R,t}} \tag{13.2-1}$$

where

σ_t is the axial tensile stress due to forces from factored actions;

f_t is the representative axial tensile strength, $f_t = f_y$;

f_y is the representative yield strength, in stress units;

$\gamma_{R,t}$ is the partial resistance factor for axial tensile strength, $\gamma_{R,t} = 1{,}05$.

The utilization of a member, U_m, under axial tension shall be calculated from Equation (13.2-2):

$$U_m = \frac{\sigma_t}{f_t / \gamma_{R,t}} \tag{13.2-2}$$

13.2.3 Axial compression

13.2.3.1 General

Tubular members subjected to axial compressive forces shall be designed to satisfy the following condition:

$$\sigma_c \leqslant \frac{f_c}{\gamma_{R,c}} \tag{13.2-3}$$

where

σ_c is the axial compressive stress due to forces from factored actions;

f_c is the representative axial compressive strength, in stress units, see 13.2.3.2;

$\gamma_{R,c}$ is the partial resistance factor for axial compressive strength, $\gamma_{R,c} = 1{,}18$.

The utilization of a member, U_m, under axial compression shall be calculated from Equation (13.2-4):

$$U_m = \frac{\sigma_c}{f_c / \gamma_{R,c}} \quad (13.2\text{-}4)$$

13.2.3.2 Column buckling

In the absence of hydrostatic pressure, the representative axial compressive strength in 13.2.3.1 for tubular members shall be the smaller of the in-plane and the out-of-plane buckling strengths determined from the following equations:

$$f_c = \left(1{,}0 - 0{,}278\lambda^2\right) f_{yc} \qquad \text{for } \lambda \leqslant 1{,}34 \quad (13.2\text{-}5)$$

$$f_c = \frac{0{,}9}{\lambda^2} f_{yc} \qquad \text{for } \lambda > 1{,}34 \quad (13.2\text{-}6)$$

$$\lambda = \sqrt{\frac{f_{yc}}{f_e}} = \frac{K\,L}{\pi\,r}\sqrt{\frac{f_{yc}}{E}} \quad (13.2\text{-}7)$$

where

- f_c is the representative axial compressive strength, in stress units;
- f_{yc} is the representative local buckling strength, in stress units, see 13.2.3.3;
- λ is the column slenderness parameter;
- f_e is the smaller of the Euler buckling strengths in the y- and z-directions, in stress units, see 13.3.3;
- E is Young's modulus of elasticity;
- K is the effective length factor in the y- or z-direction selected so that $K\,L$ is the larger of the values in the y- and z-directions, see 13.5;
- L is the unbraced length in y- or z-direction;
- r is the radius of gyration, $r = \sqrt{I / A}$;
- I is the moment of inertia of the cross-section;
- A is the cross-sectional area.

13.2.3.3 Local buckling

The representative local buckling strength, f_{yc}, in 13.2.3.2 shall be determined from:

$$f_{yc} = f_y \qquad \text{for } \frac{f_y}{f_{xe}} \leqslant 0{,}170 \quad (13.2\text{-}8)$$

$$f_{yc} = \left(1{,}047 - 0{,}274\frac{f_y}{f_{xe}}\right) f_y \qquad \text{for } 0{,}170 < \frac{f_y}{f_{xe}} \quad (13.2\text{-}9)$$

and

$$f_{xe} = 2\,C_x E\,t / D \quad (13.2\text{-}10)$$

where

f_y is the representative yield strength, in stress units;

f_{xe} is the representative elastic local buckling strength, in stress units;

C_x is the elastic critical buckling coefficient, see below;

E is Young's modulus of elasticity;

D is the outside diameter of the member;

t is the wall thickness of the member.

The theoretical value of C_x for an ideal tubular is 0,6. However, a reduced value of $C_x = 0{,}3$ should be used in Equation (13.2-10) to account for the effect of initial geometric imperfections within the tolerance limits given in Clause 21. A reduced value of $C_x = 0{,}3$ is implicit in the value of f_{xe} used in Equations (13.2-8) and (13.2-9).

13.2.4 Bending

Tubular members subjected to bending moments shall be designed to satisfy the following condition:

$$\sigma_b = \frac{M}{Z_e} \leqslant \frac{f_b}{\gamma_{R,b}} \tag{13.2-11}$$

where

σ_b is the bending stress due to forces from factored actions; when $M > M_y$, σ_b is to be considered as an equivalent elastic bending stress, $\sigma_b = M/Z_e$;

f_b is the representative bending strength, in stress units, see Equations (13.2-13) to (13.2-15);

$\gamma_{R,b}$ is the partial resistance factor for bending strength, $\gamma_{R,b} = 1{,}05$;

M is the bending moment due to factored actions;

M_y is the elastic yield moment;

Z_e is the elastic section modulus, $Z_e = \frac{\pi}{64}\left(D^4 - (D-2t)^4\right) \Big/ \left(\frac{D}{2}\right)$.

The utilization of a member, U_m, under bending moments shall be calculated from Equation (13.2-12):

$$U_m = \frac{\sigma_b}{f_b / \gamma_{R,b}} = \frac{M / Z_e}{f_b / \gamma_{R,b}} \tag{13.2-12}$$

The representative bending strength for tubular members shall be determined from:

$$f_b = \left(\frac{Z_p}{Z_e}\right) f_y \qquad \text{for } \frac{f_y D}{E t} \leqslant 0{,}0517 \tag{13.2-13}$$

$$f_b = \left[1{,}13 - 2{,}58\left(\frac{f_y D}{E t}\right)\right]\left(\frac{Z_p}{Z_e}\right) f_y \qquad \text{for } 0{,}0517 < \frac{f_y D}{E t} \leqslant 0{,}1034 \tag{13.2-14}$$

$$f_b = \left[0{,}94 - 0{,}76\left(\frac{f_y D}{E t}\right)\right]\left(\frac{Z_p}{Z_e}\right) f_y \qquad \text{for } 0{,}1034 < \frac{f_y D}{E t} \leqslant 120\frac{f_y}{E} \tag{13.2-15}$$

where, additionally,

f_y is the representative yield strength, in stress units;

D is the outside diameter of the member;

t is the wall thickness of the member;

Z_p is the plastic section modulus, $Z_p = \frac{1}{6}\left[D^3 - (D - 2t)^3\right]$.

13.2.5 Shear

13.2.5.1 Beam shear

Tubular members subjected to beam shear forces shall be designed to satisfy the following condition:

$$\tau_b = \frac{2V}{A} \leqslant \frac{f_v}{\gamma_{R,v}} \tag{13.2-16}$$

where

τ_b is the maximum beam shear stress due to forces from factored actions;

f_v is the representative shear strength, in stress units, $f_v = f_y / \sqrt{3}$;

$\gamma_{R,v}$ is the partial resistance factor for shear strength, $\gamma_{R,v} = 1{,}05$;

V is the beam shear due to factored actions, in force units;

A is the cross-sectional area.

The utilization of a member, U_m, under beam shear shall be calculated from Equation (13.2-17):

$$U_m = \frac{\tau_b}{f_v / \gamma_{R,v}} = \frac{2V / A}{f_v / \gamma_{R,v}} \tag{13.2-17}$$

13.2.5.2 Torsional shear

Tubular members subjected to torsional shear forces shall be designed to satisfy the following condition:

$$\tau_t = \frac{M_{v,t} D}{2 I_p} \leqslant \frac{f_v}{\gamma_{R,v}} \tag{13.2-18}$$

where, in addition to the definitions in 13.2.5.1,

τ_t is the torsional shear stress due to forces from factored actions;

$M_{v,t}$ is the torsional moment due to factored actions;

I_p is the polar moment of inertia, $I_p = \frac{\pi}{32}\left[D^4 - (D - 2t)^4\right]$.

The partial resistance factor, $\gamma_{R,v}$, for shear, is the same for both torsional shear and beam shear, see 13.2.5.1.

The utilization of a member, U_m, under torsional shear shall be calculated from Equation (13.2-19):

$$U_m = \frac{\tau_t}{f_v / \gamma_{R,v}} = \frac{M_{v,t} D / 2 I_p}{f_v / \gamma_{R,v}} \tag{13.2-19}$$

13.2.6 Hydrostatic pressure

13.2.6.1 Calculation of hydrostatic pressure

The effective depth at the location being checked shall be calculated taking into account the depth of the member below still water level and the effect of passing waves. The factored hydrostatic pressure (p) shall be calculated from Equation (13.2-20):

$$p = \gamma_{f,G1} \, \rho_w \, g \, H_z \tag{13.2-20}$$

where

$\gamma_{f,G1}$ is the partial action factor for permanent actions 1, see Table 9.10-1;

ρ_w is the density of the sea water which may be taken as 1 025 kg/m^3;

g is the acceleration due to gravity (m/s^2);

H_z is the effective hydrostatic head (m)

$$H_z = -z + \frac{H_w}{2} \frac{\cosh\left[k\left(d+z\right)\right]}{\cosh\left(k\,d\right)} \tag{13.2-21}$$

where

z is the depth of the member relative to still water level (measured positive upwards);

d is the still water depth to the sea floor;

H is the wave height;

k is the wave number, $k = 2\,\pi/\lambda$;

where

λ is the wave length.

For installation conditions, z shall be the maximum depth of submergence during launch, or the maximum differential head during the upending and installation sequence plus an amount to allow for deviations from the planned sequence, and $\gamma_{f,G1}$ in Equation (13.2-20) shall be replaced by $\gamma_{f,T}$, see Clause 8.

13.2.6.2 Hoop buckling

Tubular members subjected to external pressure shall be designed to satisfy the following condition:

$$\sigma_h = \frac{p\,D}{2\,t} \leqslant \frac{f_h}{\gamma_{R,h}} \tag{13.2-22}$$

where

σ_h is the hoop stress due to the forces from factored hydrostatic pressure;

p is the factored hydrostatic pressure, see 13.2.6.1;

D is the outside diameter of the member;

t is the wall thickness of the member;

f_h is the representative hoop buckling strength, in stress units, see Equations (13.2-23) to (13.2-25);

$\gamma_{R,h}$ is the partial resistance factor for hoop buckling strength, $\gamma_{R,h} = 1,25$.

For tubular members satisfying the out-of-roundness tolerances given in Annex G, f_h shall be determined from:

$$f_h = f_y \qquad \text{for } f_{he} > 2{,}44\,f_y \tag{13.2-23}$$

$$f_h = 0{,}7\left(f_{he}/f_y\right)^{0{,}4} f_y \leqslant f_y \qquad \text{for } 0{,}55\,f_y < f_{he} \leqslant 2{,}44\,f_y \tag{13.2-24}$$

$$f_h = f_{he} \qquad \text{for } f_{he} \leqslant 0{,}55\,f_y \tag{13.2-25}$$

where

f_y is the representative yield strength, in stress units;

f_{he} is the representative elastic critical hoop buckling strength, in stress units

$$f_{he} = 2C_h E t / D \tag{13.2-26}$$

where the elastic critical hoop buckling coefficient C_h is:

$$C_h = 0{,}44\ t/D \qquad \text{for } \mu \geqslant 1{,}6\ D/t \tag{13.2-27}$$

$$C_h = 0{,}44\ t/D + 0{,}21\ (D/t)^3\,\mu^4 \qquad \text{for } 0{,}825\ D/t \leqslant \mu < 1{,}6\ D/t \tag{13.2-28}$$

$$C_h = 0{,}737/(\mu - 0{,}579) \qquad \text{for } 1{,}5 \leqslant \mu < 0{,}825\ D/t \tag{13.2-29}$$

$$C_h = 0{,}80 \qquad \text{for } \mu < 1{,}5 \tag{13.2-30}$$

where μ is a geometric parameter and

$$\mu = \frac{L_r}{D}\sqrt{\frac{2D}{t}}$$

where L_r is the length of tubular between stiffening rings, diaphragms, or end connections.

For tubular members exceeding the out-of-roundness tolerances, see A.13.2.6.2.

The utilization of a member, U_m, under external pressure shall be calculated from Equation (13.2-31):

$$U_m = \frac{\sigma_h}{f_h / \gamma_{R,h}} = \frac{p\,D/2\,t}{f_h / \gamma_{R,h}} \tag{13.2-31}$$

13.2.6.3 Ring stiffener design

For $\mu \geqslant 1{,}6\,D/t$, the elastic critical hoop buckling stress is approximately equal to that of a long unstiffened tubular. Hence, to be effective, stiffening rings, if required, should be spaced such that

$$L_r < 1{,}6\sqrt{\frac{D^3}{2\,t}} \tag{13.2-32}$$

The circumferential stiffening ring size may be calculated from Equations (13.2-33) or (13.2-34) as appropriate, provided, if the yield strength of the ring stiffener is less than that of the member, that this smaller value of yield strength is used instead of f_y in Equations (13.2-23) to (13.2-25).

$$I_c = f_{he}\frac{t\,L_r\,D^2}{8\,E} \quad \text{for internal rings} \tag{13.2-33}$$

$$I_c = f_{he}\frac{t\,L_r\,D_r^2}{8\,E} \quad \text{for external rings} \tag{13.2-34}$$

Where, in addition to the definitions given in 13.2.6.2,

I_c is the required moment of inertia for the composite ring section;

L_r is the ring spacing;

D is the outside diameter of the member;

D_r is the diameter of the centroid of the composite ring section;

E is Young's modulus of elasticity.

The composite ring section may be assumed to include an effective width of the member wall of $1{,}1\sqrt{D\,t}$.

Where out-of-roundness is in excess of that permitted by Annex G, larger stiffeners can be required. In such cases the bending due to excess out-of-roundness shall be specifically investigated.

Local buckling of ring stiffeners with flanges may be excluded as a possible failure mode, provided that the following requirements are fulfilled:

$$\frac{h}{t_w} \leqslant 1{,}1\sqrt{\frac{E}{f_{y,r}}} \tag{13.2-35}$$

and

$$\frac{b}{t_f} \leqslant 0{,}6\sqrt{\frac{E}{f_{y,r}}} \tag{13.2-36}$$

where, in addition,

h is the web height;

t_w is the web thickness;

b is half the flange width of T stiffeners or the full flange width for angle stiffeners;

t_f is the flange thickness;

$f_{y,r}$ is the representative yield strength of the ring stiffeners, in stress units.

Local buckling of ring stiffeners without flanges may be excluded as a possible failure mode, provided that

$$\frac{h}{t_w} \leqslant 0{,}6\sqrt{\frac{E}{f_{y,r}}} \tag{13.2-37}$$

Ring stiffeners, including their components and whether internal or external, shall have a minimum thickness of 10 mm.

13.3 Tubular members subjected to combined forces without hydrostatic pressure

13.3.1 General

The following gives requirements for members subjected to combined forces, which give rise to global and local interactions between axial forces and bending moments, without hydrostatic pressure. Generally, the secondary moments from factored global actions and the associated bending stresses (P–Δ effects) do not need to be considered. However, when the axial member force is substantial, or when the component on which the axial force acts is very flexible, the secondary moments due to P–Δ effects from factored global actions should be taken into account.

13.3.2 Axial tension and bending

Tubular members subjected to combined axial tension and bending forces shall be designed to satisfy the following condition at all cross-sections along their length:

$$\frac{\gamma_{R,t}\,\sigma_t}{f_t} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_b} \leqslant 1{,}0 \tag{13.3-1}$$

where, in addition to the definitions in 13.2.2 and 13.2.4

$\sigma_{b,y}$ is the bending stress about the member y-axis (in-plane) due to forces from factored actions;

$\sigma_{b,z}$ is the bending stress about the member z-axis (out-of-plane) due to forces from factored actions.

The utilization of a member, U_m, under combined axial tension and bending shall be calculated from Equation (13.3-2):

$$U_m = \frac{\gamma_{R,t}\,\sigma_t}{f_t} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_b} \tag{13.3-2}$$

13.3.3 Axial compression and bending

Tubular members subjected to combined axial compression and bending forces shall be designed to satisfy the following conditions at all cross-sections along their length:

$$\frac{\gamma_{R,c}\ \sigma_c}{f_c} + \frac{\gamma_{R,b}}{f_b}\left[\left(\frac{C_{m,y}\ \sigma_{b,y}}{1-\sigma_c / f_{e,y}}\right)^2 + \left(\frac{C_{m,z}\ \sigma_{b,z}}{1-\sigma_c / f_{e,z}}\right)^2\right]^{0,5} \leqslant 1,0 \tag{13.3-3}$$

and

$$\frac{\gamma_{R,c}\ \sigma_c}{f_{yc}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_b} \leqslant 1,0 \tag{13.3-4}$$

where, in addition to the definitions given in 13.2.3, 13.2.4 and 13.3.2,

$C_{m,y}$, $C_{m,z}$ are the moment reduction factors corresponding to the member y- and z-axes, respectively (see 13.5);

$f_{e,y}$, $f_{e,z}$ are the Euler buckling strengths corresponding to the member y- and z-axes, respectively, in stress units

$$f_{e,y} = \frac{\pi^2 E}{\left(K_y\ L_y / r\right)^2} \tag{13.3-5}$$

$$f_{e,z} = \frac{\pi^2 E}{\left(K_z\ L_z / r\right)^2} \tag{13.3-6}$$

where

K_y, K_z are the effective length factors for the y- and z-directions, respectively, see 13.5;

L_y, L_z are the unbraced lengths in the y- and z-directions, respectively.

The utilization of a member, U_m, under axial compression and bending shall be the larger value calculated from Equations (13.3-7) and (13.3-8):

$$U_m = \frac{\gamma_{R,c}\ \sigma_c}{f_c} + \frac{\gamma_{R,b}}{f_b}\left[\left(\frac{C_{m,y}\ \sigma_{b,y}}{1-\sigma_c / f_{e,y}}\right)^2 + \left(\frac{C_{m,z}\ \sigma_{b,z}}{1-\sigma_c / f_{e,z}}\right)^2\right]^{0,5} \tag{13.3-7}$$

$$U_m = \frac{\gamma_{R,c}\ \sigma_c}{f_{yc}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_b} \tag{13.3-8}$$

13.3.4 Piles

Overall column buckling is normally not a problem in the design of pile segments below the sea floor because the surrounding soils inhibit overall column buckling. However, whenever laterally loaded piles are subjected to significant axial actions, the secondary moments (P–Δ effects) should be considered in stress computations. An effective method of analysis is to model the pile as a beam-column on an elastic foundation. When such an analysis is used, the pile segment should be designed to satisfy Equation (13.3-4), except that $\sigma_{b,y}$ and $\sigma_{b,z}$ in this formula should include the stresses from the secondary moments (P–Δ effects) computed from factored actions.

13.4 Tubular members subjected to combined forces with hydrostatic pressure

13.4.1 General

A tubular member below the water line is subjected to hydrostatic pressure unless it has been flooded. Flooding is normally only used for a structure's legs in order to assist in upending and placement and for pile installation. Even where members are flooded in the in-place condition, they can be subjected to hydrostatic pressure during launch and installation. The effects of hydrostatic pressure shall be taken into account when conducting member checks, including the axial components of such pressure (i.e. capped-end actions). When conducting an analysis of the axial components of hydrostatic pressure, such action effects can be taken directly into account during the analysis or can be included subsequently. The formulations presented in 13.4 allow either approach for accounting for the axial effects of hydrostatic pressure to be used.

When checking tubular members subjected to hydrostatic pressure, four checks are required:

a) check for hoop buckling under hydrostatic pressure alone, Equation (13.2-22);

b) check for tensile yielding when the combination of action effects, including those due to capped-end forces, results in tension in the member, 13.4.2;

c) check for compression yielding and local buckling when the combination of action effects, including those due to capped-end forces, results in compression in the member, 13.4.3;

d) check for column buckling when the action effects, excluding those due to capped-end forces, result in compression in the member, 13.4.3.

For analyses using factored actions that include capped-end actions:

$\sigma_{t,c}$ is the axial tensile stress due to forces from factored actions;

$\sigma_{c,c}$ is the axial compressive stress due to forces from factored actions.

For analyses using factored actions that do not include the capped-end actions:

$$\sigma_{t,c} = \sigma_t - \sigma_q \qquad \text{if } \sigma_t \geqslant \sigma_q \tag{13.4-1}$$

$$\sigma_{c,c} = \sigma_q - \sigma_t \qquad \text{if } \sigma_t < \sigma_q \tag{13.4-2}$$

$$\sigma_{c,c} = \sigma_c + \sigma_q \tag{13.4-3}$$

where

σ_t is the axial tensile stress due to forces from factored actions without capped-end actions;

σ_c is the axial compressive stress due to forces from factored actions without capped-end actions;

σ_q is the compressive axial stress due to the capped-end hydrostatic actions calculated using the value of pressure from Equation (13.2-20).

NOTE 1 In some circumstances, the use of factored actions leads to conditions in which $\sigma_{t,c}$ is tensile, whereas under unfactored actions $\sigma_{t,c}$ is compressive. These cases usually occur for relatively low values of $\sigma_{t,c}$ and the error is not considered to be significant.

The capped-end stresses (σ_q) may be approximated as half the hoop stress due to forces from factored hydrostatic pressure, i.e.

$$\sigma_q = 0{,}5\ \sigma_h \tag{13.4-4}$$

NOTE 2 In accordance with 13.1, σ_q always has a positive value.

In reality, the magnitude of these stresses depends on the restraint on the member provided by the rest of the structure and its value can be more or less than 0,5 σ_h. The approximation 0,5 σ_h may be replaced by a stress computed from a more rigorous analysis, using factored actions.

When an analysis uses factored actions that include capped-end actions, σ_c for net compression cases may be approximated by

$$\sigma_c = \sigma_q - \sigma_{t,c} \qquad \text{if } \sigma_{t,c} < \sigma_q \tag{13.4-5}$$

$$\sigma_c = \sigma_{c,c} - \sigma_q \qquad \text{if } \sigma_{c,c} > \sigma_q \tag{13.4-6}$$

13.4.2 Axial tension, bending and hydrostatic pressure

Tubular members subjected to combined axial tension, bending and hydrostatic pressure shall be designed to satisfy the following requirements at all cross-sections along their length.

$$\frac{\gamma_{R,t}\sigma_{t,c}}{f_{t,h}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_{b,h}} \leqslant 1{,}0 \tag{13.4-7}$$

where, in addition to the definitions in 13.2, 13.3 and 13.4.1:

$f_{t,h}$ is the representative axial tensile strength in the presence of external hydrostatic pressure, in stress units

$$f_{t,h} = f_y\left(\sqrt{1+0{,}09B^2 - B^{2\eta}} - 0{,}3B\right) \tag{13.4-8}$$

$f_{b,h}$ is the representative bending strength in the presence of external hydrostatic pressure, in stress units

$$f_{b,h} = f_b\left(\sqrt{1+0{,}09B^2 - B^{2\eta}} - 0{,}3B\right) \tag{13.4-9}$$

and

$$B = \frac{\gamma_{R,h}\sigma_h}{f_h}, \qquad B \leqslant 1{,}0 \tag{13.4-10}$$

$$\eta = 5 - 4\frac{f_h}{f_y} \tag{13.4-11}$$

The utilization of a member, U_m, under axial tension, bending and hydrostatic pressure shall be calculated from Equation (13.4-12):

$$U_m = \frac{\gamma_{R,t}\ \sigma_{t,c}}{f_{t,h}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_{b,h}} \tag{13.4-12}$$

13.4.3 Axial compression, bending and hydrostatic pressure

Tubular members subjected to combined axial compression, bending and hydrostatic pressure shall be designed to satisfy the following requirements at all cross-sections along their length.

$$\frac{\gamma_{R,c}\ \sigma_{c,c}}{f_{yc}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_{b,h}} \leqslant 1{,}0 \tag{13.4-13}$$

$$\frac{\gamma_{R,c}\ \sigma_c}{f_{c,h}} + \frac{\gamma_{R,b}}{f_{b,h}}\left[\left(\frac{C_{m,y}\ \sigma_{b,y}}{1-\sigma_c/f_{e,y}}\right)^2 + \left(\frac{C_{m,z}\ \sigma_{b,z}}{1-\sigma_c/f_{e,z}}\right)^2\right]^{0,5} \leqslant 1{,}0 \tag{13.4-14}$$

where, additionally:

$f_{c,h}$ is the representative axial compressive strength in the presence of external hydrostatic pressure, in stress units

$$f_{c,h} = \frac{1}{2} f_{yc}\left[\left(1{,}0 - 0{,}278\lambda^2\right) - \frac{2\sigma_q}{f_{yc}} + \sqrt{\left(1{,}0 - 0{,}278\lambda^2\right)^2 + 1{,}12\lambda^2\frac{\sigma_q}{f_{yc}}}\right] \quad \text{for } \lambda \leqslant 1{,}34\sqrt{\left(1-\frac{2\sigma_q}{f_{yc}}\right)^{-1}} \tag{13.4-15}$$

$$f_{c,h} = \frac{0{,}9}{\lambda^2} f_{yc} \quad \text{for } \lambda > 1{,}34\sqrt{\left(1-\frac{2\sigma_q}{f_{yc}}\right)^{-1}} \tag{13.4-16}$$

If the maximum combined compressive stress, $\sigma_x = \sigma_b + \sigma_{c,c}$, and the representative elastic local buckling strength, f_{xe}, exceed the limits given below, then Equation (13.4-18) shall also be satisfied:

$$\sigma_x > 0{,}5\frac{f_{he}}{\gamma_{R,h}} \text{ and } \frac{f_{xe}}{\gamma_{R,c}} > 0{,}5\frac{f_{he}}{\gamma_{R,h}} \tag{13.4-17}$$

$$\frac{\sigma_x - 0{,}5 f_{he}/\gamma_{R,h}}{f_{xe}/\gamma_{R,c} - 0{,}5 f_{he}/\gamma_{R,h}} + \left(\frac{\gamma_{R,h}\sigma_h}{f_{he}}\right)^2 \leqslant 1{,}0 \tag{13.4-18}$$

where

f_{he} is the representative elastic critical hoop buckling strength defined in 13.2.6.2;

f_{xe} is the representative elastic local buckling strength defined in 13.2.3.3.

The utilization of a member, U_m, under axial compression, bending and hydrostatic pressure shall be the largest value calculated from Equations (13.4-19), (13.4-20) and (13.4-21):

$$U_m = \frac{\gamma_{R,c}\ \sigma_{c,c}}{f_{yc}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_{b,h}} \quad \text{when Equation (13.4-13) applies} \tag{13.4-19}$$

$$U_m = \frac{\gamma_{R,c}\ \sigma_c}{f_{c,h}} + \frac{\gamma_{R,b}}{f_{b,h}}\left[\left(\frac{C_{m,y}\ \sigma_{b,y}}{1-\sigma_c/f_{e,y}}\right)^2 + \left(\frac{C_{m,z}\ \sigma_{b,z}}{1-\sigma_c/f_{e,z}}\right)^2\right]^{0,5} \quad \text{when Equation (13.4-14) applies} \tag{13.4-20}$$

$$U_m = \frac{\sigma_x - 0{,}5 f_{he}/\gamma_{R,h}}{f_{xe}/\gamma_{R,c} - 0{,}5 f_{he}/\gamma_{R,h}} + \left(\frac{\gamma_{R,h}\ \sigma_h}{f_{he}}\right)^2 \quad \text{when Equation (13.4-18) applies} \tag{13.4-21}$$

13.5 Effective lengths and moment reduction factors

The effective lengths and moment reduction factors may be determined using a rational analysis that includes joint flexibility and side-sway. In lieu of such a rational analysis, values of effective length factors (K) and moment reduction factors (C_m) may be taken from Table 13.5-1. These factors

a) do not apply to cantilever members, and

b) assume both member ends are rotationally restrained in both planes of bending (see A.13.5).

NOTE Examples of the use of rational analysis can be found in the relevant references cited in Annex A.

Lengths to which the effective length factors K are applied are normally measured from centreline to centreline of the end joints. However, for members framing into legs, the following modified lengths may be used, provided that no interaction between the buckling of members and legs affects the utilization of the legs:

— face-of-leg to face-of-leg for main diagonal braces;

— face-of-leg to centreline of end joint for K-braces.

Lower K factors than those according to Table 13.5-1 may be used provided they are supported by more rigorous analysis.

Table 13.5-1 — Effective length and moment reduction factors for member strength checking

Structural component			K	C_m [a]
Topsides legs				
	Braced		1,0	1)
	Portal (unbraced)		K [b]	1)
Structure legs and piling				
	Grouted composite section		1,0	3)
	Ungrouted legs		1,0	3)
	Ungrouted piling between shim points		1,0	2)
Structure brace members				
	Primary diagonals and horizontals		0,7	2) or 3)
	K- braces [c]		0,7	2) or 3)
	X -braces			
		Longer segment length [c]	0,8	2) or 3)
		Full length [d]	0,7	2) or 3)
Secondary horizontals			0,7	2) or 3)

[a] C_m values for the three cases defined in this table are as follows:

1) 0,85;
2) for members with no transverse loading, other than self weight,

   $C_m = 0{,}6 - 0{,}4 \times M_1/M_2$

   where M_1/M_2 is the ratio of smaller to larger moments at the ends of the unbraced portion of the member in the plane of bending under consideration;

   M_1/M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature.

   C_m shall not be larger than 0,85;
3) for members with transverse loading, other than self weight,

   $C_m = 1{,}0 - 0{,}4 \times (\sigma_c/f_e)$, or 0,85, whichever is less,

   and $f_e = f_{ey}$ or f_{ez} as appropriate.

[b] See effective length alignment chart in A.13.5. This may be modified to account for conditions different from those assumed in the development of the chart.

[c] For either in-plane or out-of-plane effective lengths, at least one pair of members framing into a K- or X-joint shall be in tension, if the joint is not braced out-of-plane.

[d] When all members are in compression and the joint is not braced out-of-plane.

13.6 Conical transitions

13.6.1 General

The following requirements apply to the design of concentric cone frusta between tubular sections and particularly to the junction between cylindrical and conical sections. They may also be applied to conical transitions on tubular joints.

The angle of the transition, α, should be less than 30º, see Figure 13.6-1.

13.6.2 Design stresses

13.6.2.1 Equivalent axial stress in conical transitions

The equivalent axial stress (meridional stress) at any section s-s within a cone (see Figure 13.6-1) can be determined from Equation (13.6-1):

$$\sigma_{\mathrm{a,eq}} = \frac{\sigma_{\mathrm{a,c}} + \sigma_{\mathrm{b,c}}}{\cos\alpha} \tag{13.6-1}$$

where

$\sigma_{\mathrm{a,\,eq}}$ is the equivalent axial stress at section s-s;

$\sigma_{\mathrm{a,\,c}}$ is the axial stress at section s-s due to global axial forces from factored actions;

$$\sigma_{\mathrm{a,c}} = \frac{P_{\mathrm{s}}}{\pi(D_{\mathrm{s}} - t_{\mathrm{c}}\cos\alpha)t_{\mathrm{c}}} \tag{13.6-2}$$

$\sigma_{\mathrm{b,\,c}}$ is the bending stress at section s-s due to global bending moments from factored actions;

$$\sigma_{\mathrm{b,c}} = \frac{4\,M_{\mathrm{s}}}{\pi(D_{\mathrm{s}} - t_{\mathrm{c}}\cos\alpha)^2 t_{\mathrm{c}}} \tag{13.6-3}$$

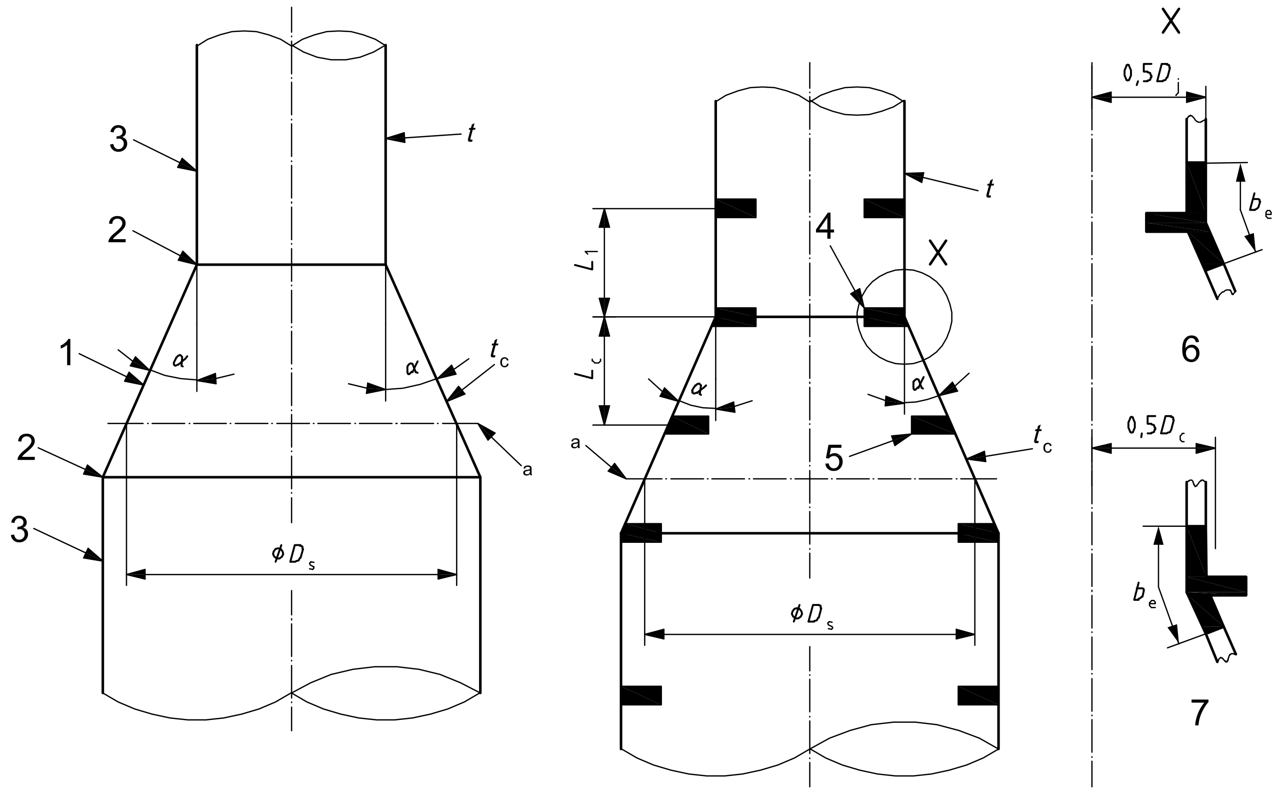
D_{s} is the outer diameter of the cone at section s-s;

t_{c} is the thickness of the cone at section s-s;

α is the slope angle of the cone (see Figure 13.6-1);

P_{s} is the axial force at section s-s due to factored actions;

M_{s} is the bending moment at section s-s due to factored actions.



a) **Unstiffened** b) **Internally stiffened**

Key

1 cone
2 junction between tubular and conical section
3 tubular
4 junction stiffener
5 intermediate stiffener
6 typical internal junction stiffener
7 alternative external junction stiffener

α slope angle of cone
b_e effective shell width (acting with stiffener)
D_c diameter to centroid of composite external ring section
D_j outer diameter at junction
D_s outer cone diameter at section s-s
L_1 distance from junction to first ring stiffener in tubular member
L_c distance from junction to first ring stiffener in cone section
t tubular member thickness at the junction
t_c cone thickness

[a] Section s-s under consideration.

Figure 13.6-1 — Typical unstiffened and stiffened conical transition

13.6.2.2 Local stresses at unstiffened junctions

13.6.2.2.1 Stress generation

The transfer of longitudinal force between a cylindrical section and a cone results in both a radial and an axial component at the junction, which generate

a) local bending stresses in the tubular and the cone at the junction, and

b) hoop stresses in both the tubular and the cone at the junction.

13.6.2.2.2 Bending stresses

In lieu of a detailed analysis, the local bending stress at each side of an unstiffened tubular-cone junction may be estimated from Equations (13.6-4) and (13.6-5):

$$\sigma_{\mathrm{b,jt}} = \frac{0{,}6t\sqrt{D_{\mathrm{j}}\left(t+t_{\mathrm{c}}\right)}}{t^2}\left(\sigma_{\mathrm{a,t}}+\sigma_{\mathrm{b,t}}\right)\tan\alpha \tag{13.6-4}$$

and

$$\sigma_{\mathrm{b,jc}} = \frac{0{,}6\,t\sqrt{D_{\mathrm{j}}\left(t+t_{\mathrm{c}}\right)}}{t_{\mathrm{c}}^2}\left(\sigma_{\mathrm{a,t}}+\sigma_{\mathrm{b,t}}\right)\tan\alpha \tag{13.6-5}$$

where

$\sigma_{\mathrm{b,jt}}$ is the local bending stress at the tubular side of the junction;

$\sigma_{\mathrm{b,jc}}$ is the local bending stress at the cone side of the junction;

D_{j} is the outer diameter at the junction (see Figure 13.6-1);

t is the wall thickness of the tubular member at the junction;

t_{c} is the wall thickness of the cone at the junction;

$\sigma_{\mathrm{a,\,t}}$ is the axial stress in the tubular section at the junction due to global axial forces from factored actions;

$\sigma_{\mathrm{b,\,t}}$ is the bending stress in the tubular section at the junction due to global bending moments from factored actions;

α is the slope angle of the cone.

13.6.2.2.3 Hoop stresses

The hoop stresses at an unstiffened tubular-cone junction due to unbalanced radial line forces may be estimated using Equations (13.6-6) and (13.6-7):

$$\sigma_{\mathrm{h,t}} = 0{,}45\sqrt{\frac{D_{\mathrm{j}}}{t}}\times\left(\sigma_{\mathrm{a,t}}+\sigma_{\mathrm{b,t}}\right)\tan\alpha \tag{13.6-6}$$

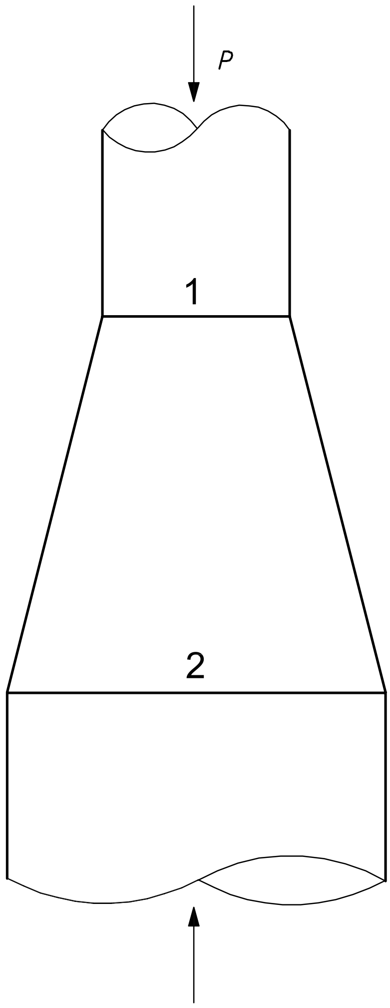
$$\sigma_{\mathrm{h,c}} = 0{,}45\sqrt{\frac{D_{\mathrm{j}}}{t}}\times\frac{t}{t_{\mathrm{c}}}\left(\sigma_{\mathrm{a,t}}+\sigma_{\mathrm{b,t}}\right)\tan\alpha \tag{13.6-7}$$

where, in addition to previous variables,

$\sigma_{\mathrm{h,t}}$ is the hoop stress at the tubular side of the junction;

$\sigma_{\mathrm{h,c}}$ is the hoop stress at the cone side of the junction.

At the smaller diameter junction, the hoop stress is tensile (or compressive) when ($\sigma_{\mathrm{a,\,t}}+\sigma_{\mathrm{b,\,t}}$) is tensile (or compressive). Similarly, the hoop stress at the larger diameter junction is tensile (or compressive) when ($\sigma_{\mathrm{a,\,t}}+\sigma_{\mathrm{b,\,t}}$) is compressive (or tensile); see Figure 13.6-2.



Key

1 area of hoop compression

2 area of hoop tension

Figure 13.6-2 — Hoop tension and hoop compression at cone junctions due to axial forces

13.6.3 Strength requirements without external hydrostatic pressure

13.6.3.1 General

The properties of a conical transition shall be chosen to satisfy the requirements given in 13.6.3 and 13.6.4 for combined axial and bending stresses at each end of the cone and along the cone transition. Conical sections may generally be checked as tubular sections with equivalent diameters and actual thicknesses.

13.6.3.2 Local buckling within conical transition

For local buckling under combined axial compression and bending, the following equation shall be satisfied at all sections within the cone:

$$\sigma_{a,eq} \leqslant \frac{f_{yc}}{\gamma_{R,c}} \tag{13.6-8}$$

where

$\sigma_{a,eq}$ is the equivalent axial stress at any section s-s, see 13.6.2.1;

f_{yc} is the representative local buckling strength of a cone, in stress units;

$\gamma_{R,c}$ is the partial resistance factor for axial compressive strength (see 13.2.3.1).

For cones with slope angle $\alpha \leqslant 30°$, f_{yc} may be determined using Equations (13.2-8) to (13.2-10) in which f_y is the representative yield strength of the cone section being checked and t and D respectively are replaced by the wall thickness of the cone, t_c, and an equivalent diameter, D_e, both at the section under consideration. D_e is given by

$$D_e = \frac{D_s}{\cos\alpha} \tag{13.6-9}$$

where D_s and α are defined in 13.6.2.1. If $\alpha > 30°$, Equation (13.6-9) is not applicable.

The utilization of a cone, U_m, under local buckling shall be calculated from Equation (13.6-10):

$$U_m = \gamma_{R,c} \frac{\sigma_{a,eq}}{f_{yc}} \tag{13.6-10}$$

13.6.3.3 Junction yielding

The requirements for junction yielding only apply to cases where the hoop stresses $\sigma_{h,t}$ and $\sigma_{h,c}$ are tensile. Yielding at a junction shall be checked on both the tubular and the cone side of the junction using Equation (13.6-11) or Equation (13.6-12), as appropriate.

For net axial tension, i.e. when σ_{max} is tensile:

$$\sqrt{\sigma_{max}^2 + \left(\sigma_j\right)^2 - \sigma_j \cdot \sigma_{max}} \leqslant \frac{f_y}{\gamma_{R,t}} \tag{13.6-11}$$

For net axial compression, i.e. when σ_{max} is compressive:

$$\sqrt{\sigma_{max}^2 + \left(\sigma_j\right)^2 + \sigma_j \cdot \left|\sigma_{max}\right|} \leqslant \frac{f_y}{\gamma_{R,t}} \tag{13.6-12}$$

where

σ_{max} is the maximum axial tensile stress at the junction;

$\sigma_{max} = \sigma_{a,t} + \sigma_{b,t} + \sigma_{b,jt}$ for checking yielding on the tubular side of the junction,

$\sigma_{max} = \dfrac{\sigma_{a,c} + \sigma_{b,c}}{\cos\alpha} + \sigma_{b,jc}$ for checking yielding on the cone side of the junction,

(see 13.6.2.1 and 13.6.2.2 for definitions of the stress components),

σ_j is the hoop stress at the junction

$\sigma_j = \sigma_{h,t}$ for checking yielding on the tubular side of the junction;

$\sigma_j = \sigma_{h,c}$ for checking yielding on the cone side of the junction;

f_y is the representative yield strength of the tubular or cone section being checked;

$\gamma_{R,t}$ is the partial resistance factor for axial tensile strength (see 13.2.2).

The utilization of a junction of a conical transition, U_m, for junction yielding shall be calculated from Equation (13.6-13) or Equation (13.6-14), as appropriate:

$$U_m = \gamma_{R,t} \frac{\sqrt{\sigma_{max}^2 + \left(\sigma_j\right)^2 - \sigma_j \cdot \sigma_{max}}}{f_y} \qquad \text{when } \sigma_{max} \text{ is tensile} \qquad (13.6\text{-}13)$$

$$U_m = \gamma_{R,t} \frac{\sqrt{\sigma_{max}^2 + \left(\sigma_j\right)^2 + \sigma_j \cdot \left|\sigma_{max}\right|}}{f_y} \qquad \text{when } \sigma_{max} \text{ is compressive} \qquad (13.6\text{-}14)$$

13.6.3.4 Junction buckling

The requirements for junction buckling only apply to cases where the hoop stress, σ_j, is compressive, and shall be checked on both the tubular and the cone side of the junction using Equation (13.6-15) or Equations (13.6-19) and (13.6-20), as appropriate.

For net axial tension, i.e. when σ_{max} is tensile, the following requirement shall be satisfied:

$$A^2 + B^{2\eta} + 2\nu A \times B \leqslant 1{,}0 \qquad (13.6\text{-}15)$$

where, in addition to the definitions given in 13.6.3.3,

$$A = \frac{\gamma_{R,t}\ \sigma_{max}}{f_y} \qquad (13.6\text{-}16)$$

$$B = \frac{\gamma_{R,h}\sigma_j}{f_h} \qquad (13.6\text{-}17)$$

$\gamma_{R,h}$ is the partial resistance factor for hoop buckling strength (see 13.2.6.2);

f_h is the representative hoop buckling strength, in stress units, calculated using Equations (13.2-23) to (13.2-25), with $f_{he} = 0{,}4\ E\ t/D_j$;

σ_j is the positive absolute value of the hoop compression;

ν is Poisson's ratio, $\nu = 0{,}3$;

η is as defined in Equation (13.4-11);

The utilization of a junction of a conical transition, U_m, for junction buckling when σ_{max} is tensile shall be calculated from Equation (13.6-18):

$$U_m = A^2 + B^{2\eta} + 2\nu A \times B \quad (13.6\text{-}18)$$

For net axial compression, i.e. when σ_{max} is compressive, the following requirements shall be satisfied:

$$\sigma_{max} \leqslant \frac{f_{yc}}{\gamma_{R,c}} \quad (13.6\text{-}19)$$

and

$$\sigma_j \leqslant \frac{f_h}{\gamma_{R,h}} \quad (13.6\text{-}20)$$

where

f_{yc} is the representative local buckling strength of the section being checked, in stress units (see 13.2.3.3); for cones with slope angle $\alpha \leqslant 30°$, f_{yc} may be calculated in accordance with 13.6.3.2;

$\gamma_{R,c}$ is the partial resistance factor for axial compressive strength (see 13.2.3.1);

f_h is the representative hoop buckling strength, in stress units, calculated using Equations (13.2-23) to (13.2-25), with $f_{he} = 0{,}4\ E\ t/D_j$;

σ_j is the positive absolute value of the hoop compression.

The utilization of a junction of a conical transition, U_m, for junction buckling when σ_{max} is compressive shall be the larger value calculated from Equations (13.6-21) and (13.6-22):

$$U_m = \frac{\sigma_{max}}{f_{yc}/\gamma_{R,c}} \quad (13.6\text{-}21)$$

$$U_m = \frac{\sigma_j}{f_h/\gamma_{R,h}} \quad (13.6\text{-}22)$$

13.6.3.5 Junction fatigue

An unstiffened junction of a conical transition shall satisfy the fatigue requirements of Clause 16, with the following stress concentration factors, C:

$$C_t = 1 + \frac{\sigma_{b,jt}}{\sigma_{a,t} + \sigma_{b,t}} \quad \text{on the tubular side} \quad (13.6\text{-}23)$$

and

$$C_c = 1 + \frac{\sigma_{b,jc}}{\sigma_{a,c} + \sigma_{b,c}} \quad \text{on the cone side} \quad (13.6\text{-}24)$$

where the variables are as defined in 13.6.2.1 and 13.6.2.2.1.

For equal tubular member and cone wall thicknesses, $\sigma_{b,\,jt}$ approximates to $\sigma_{b,\,jc}$ and, similarly, $\sigma_{a,\,c}$ and $\sigma_{b,\,c}$ at the junction approximate to $\sigma_{a,\,t}$ and $\sigma_{b,\,t}$, respectively; so that Equations (13.6-23) and (13.6-24) reduce to

$$C_t = C_c = 1 + 0{,}6\sqrt{\frac{2D_j}{t}}\tan\alpha \qquad (13.6\text{-}25)$$

13.6.4 Strength requirements with external hydrostatic pressure

13.6.4.1 Hoop buckling

Unstiffened conical transitions or cone segments between stiffening rings with a slope angle $\alpha \leqslant 30°$, may be designed for hoop buckling as equivalent tubulars in accordance with 13.4. The equivalent diameter is $D/\cos\alpha$, where D is the diameter at the larger end of the cone or segment. An equivalent axial stress shall be used in the equations in 13.4; the equivalent axial stress shall be calculated using Equation (13.6-1), substituting $\sigma_{a,c}$ for $\sigma_{t,c}$ or $\sigma_{c,c}$, σ_t, or σ_c, as appropriate. The length of the cone shall be taken as the maximum distance between adjacent rings for a ring-stiffened cone transition or the maximum unstiffened distance, including the tubular members on both ends of the cone, for an unstiffened conical transition. The tubular members on both ends of a ring-stiffened cone shall also be checked against the requirements of 13.4.

13.6.4.2 Junction yielding and buckling

The net hoop stress ($\sigma_{h,\,j}$) at a junction of a conical transition is given by

$$\sigma_{h,j} = \sigma_j + \sigma_h \qquad (13.6\text{-}26)$$

where

σ_j is the hoop stress at the junction due to unbalanced radial forces as defined in 13.6.2.2.3 and 13.6.3.3;

σ_h is the hoop stress due to forces from the factored external hydrostatic pressure (see 13.2.6).

When $\sigma_{h,\,j}$ is tensile, Equations (13.6-11) to (13.6-14) in 13.6.3.3 shall be satisfied using $\sigma_{h,\,j}$ instead of σ_j. When $\sigma_{h,\,j}$ is compressive, Equations (13.6-15) to (13.6-22) in 13.6.3.4 shall be satisfied using $\sigma_{h,\,j}$ instead of σ_j, while the representative hoop buckling strength f_h shall be determined in accordance with 13.2.6.2 using the equivalent diameter as described in 13.6.3.2.

13.6.5 Ring design

13.6.5.1 General

A tubular-cone junction that does not satisfy the requirements of 13.6.3 or 13.6.4 can be strengthened either by increasing the tubular and cone thicknesses at the junction or by providing a stiffening ring at the junction. Where rings are provided, their dimensions shall satisfy Equations (13.2-35) and (13.2-36) or (13.2-37), as appropriate.

13.6.5.2 Junction rings without external hydrostatic pressure

If stiffening rings are required in the absence of hydrostatic pressure, their section properties shall satisfy both the following requirements:

$$A_c = \frac{t\,D_j}{f_y}(\sigma_a + \sigma_b)\tan\alpha \qquad (13.6\text{-}27)$$

$$I_c = \frac{t\,D_j\,D_c^2}{8E}(\sigma_a + \sigma_b)\tan\alpha \qquad (13.6\text{-}28)$$

where

t is the thickness of the tubular member at the junction;

σ_a is the larger of $\sigma_{a,\,t}$ (see 13.6.2.2.2) and $\sigma_{a,\,c}$ (see 13.6.2.1);

σ_b is the larger of $\sigma_{b,\,t}$ (see 13.6.2.2.2) and $\sigma_{b,\,c}$ (see 13.6.2.1);

D_j is the outer diameter at the junction (see Figure 13.6-1);

D_c for external rings, is the diameter to centroid of composite ring section (see Figure 13.6-1);

D_c for internal rings, is taken as D_j;

α is the slope angle of the conical transition, see Figure 13.6-1;

A_c is the cross-sectional area of the composite ring section;

I_c is the moment of inertia of the composite ring section.

In computing A_c and I_c, the effective width of shell wall acting as a flange for the composite ring section (see Figure 13.6-1) may be computed from Equation (13.6-29):

$$b_e = 0{,}55\left(\sqrt{D_j\,\mathrm{t}} + \sqrt{D_j\,t_c}\right) \tag{13.6-29}$$

Where out-of-roundness is in excess of that permitted by Annex G, larger stiffeners can be required. The bending due to excess out-of-roundness shall be specifically investigated.

13.6.5.3 Junction rings with external hydrostatic pressure

If stiffening rings are required in the presence of external hydrostatic pressure, their section properties and spacing shall be chosen to satisfy the following equations:

$$I_{c,T} \geqslant I_c + I_{c,h} \tag{13.6-30}$$

$$L_1 \leqslant 1{,}13\sqrt{\frac{D_j^3}{t}} \tag{13.6-31}$$

$$L_c \leqslant 1{,}13\sqrt{\frac{D_e^3}{t}} \tag{13.6-32}$$

where

$I_{c,T}$ is the moment of inertia of the composite ring section in the presence of external hydrostatic pressure;

I_c is the moment of inertia of the composite ring section in the absence of external hydrostatic pressure (see 13.6.5.2);

$I_{c,\,h}$ is the additional moment of inertia of the composite ring section required for external hydrostatic pressure

$$I_{c,h} = \frac{{D_c}^2}{16E}\left(t\,L_1\,f_{he} + \frac{t_c\,L_c\,f_{hec}}{\cos^2\alpha}\right) \tag{13.6-33}$$

where

D_c is as defined in 13.6.5.2;

t is the wall thickness of the tubular member at the junction under consideration;

t_c is the wall thickness of the cone at the junction under consideration;

α is the slope angle of the conical transition (see Figure 13.6-1);

E is Young's modulus of elasticity;

L_1 is the distance from the junction to the first stiffening ring in the tubular section (see Figure 13.6-1);

L_c is the distance from the junction to the first stiffening ring in the cone, measured along the axis of the cone (see Figure 13.6-1);

f_{he} is the representative elastic critical hoop buckling stress of the tubular [see Equation (13.2-26)];

f_{hec} is the representative elastic critical hoop buckling stress of the conical transition treated as an equivalent tubular of diameter D_e;

D_e is the largest of the equivalent diameters for the bay of the conical transition adjacent to the junction under consideration [see Equation (13.6-9)].

The effective width of shell wall acting as a flange for the composite ring section (see Figure 13.6-1) may be computed from Equation (13.6-29).

A junction ring is not required for hydrostatic collapse if Equation (13.2-22) is satisfied using $\sigma_{h,\,j}$ from 13.6.4.2. In satisfying Equation (13.2-22), the elastic hoop buckling stress, f_{he}, should be computed using Equation (13.2-26) with $C_h = 0{,}44\ t \cos\alpha / D_j$.

Where out-of-roundness is in excess of that permitted by Annex G, larger stiffeners can be required. The bending due to excess out-of-roundness shall be specifically investigated.

13.6.5.4 Intermediate stiffening rings

If required, circumferential stiffening rings within conical transitions shall be designed using Equation (13.2-33) or Equation (13.2-34), as appropriate, with the following modifications:

— for internal rings, diameter D shall be taken as being equal to the equivalent diameter, D_e, from Equation (13.6-9);

— for external rings, diameter D_r shall be taken as being equal to the diameter, D_c, to the centroid of the composite ring section, see Figure 13.6-1;

— the wall thickness, t, shall be taken as being equal to the cone thickness, t_c, at the position of the stiffening ring;

— the spacing, L_r, shall be taken as the average distance along the axis of the cone between adjacent rings;

— f_{he} shall be taken as the average of the representative elastic critical hoop buckling stresses for the two adjacent bays calculated using equivalent diameters, with the equivalent diameters based on the largest diameter for each of the adjacent bays.

13.7 Dented tubular members

13.7.1 General

Neither the effects of external hydrostatic pressure nor the presence of cracks have been considered in the derivation of the equations of 13.7. Such effects shall be considered on a case-by-case basis. Fatigue problems can arise at dent locations and shall also be considered.

The limits of calibration for the formulae in this subclause are a dent depth, h, of $h \leqslant 0{,}3\ D$ and $h \leqslant 10\ t$. The effects of dent depths in excess of either of these values shall be examined by other means.

13.7.2 Dented tubular members subjected to tension, compression, bending or shear

13.7.2.1 General

Dented tubular members subjected independently to axial tension, axial compression, bending or shear shall be assessed using the strength and stability requirements specified in 13.7.2.2 to 13.7.2.5.

13.7.2.2 Axial tension

Dented tubular members subjected to axial tensile forces shall satisfy the following condition:

$$\sigma_{t,d} \leqslant \frac{f_y}{\gamma_{R,t,d}} \tag{13.7-1}$$

where

- $\sigma_{t,d}$ is the axial tensile stress due to forces from factored actions on the undamaged cross-section, $\sigma_{t,d} = T/A$;
- T is the member axial tensile force;
- A is the cross-sectional area of the undamaged section;
- f_y is the representative yield strength, in stress units;
- $\gamma_{R,t,d}$ is the partial resistance factor for axial tensile strength for dented members, $\gamma_{R,t,d} = 1{,}05$.

The utilization of a dented member, $U_{m,d}$, under axial tension shall be calculated from Equation (13.7-2):

$$U_{m,d} = \frac{\sigma_{t,d}}{f_y / \gamma_{R,t,d}} \tag{13.7-2}$$

13.7.2.3 Axial compression

13.7.2.3.1 General

Dented tubular members subjected to axial compressive forces shall satisfy the following condition:

$$\sigma_{c,d} \leqslant \frac{f_{c,d}}{\gamma_{R,c,d}} \tag{13.7-3}$$

where

- $\sigma_{c,d}$ is the axial compressive stress due to forces from factored actions on the undamaged cross-section, $\sigma_{c,d} = P/A$;

P is the member axial compressive force;

$f_{c,d}$ is the representative axial compressive strength of dented members, in stress units, see 13.7.2.3.2;

$\gamma_{R,c,d}$ is the partial resistance factor for axial compressive strength of dented members, $\gamma_{R,c,d} = 1{,}18$.

The utilization of a dented member, $U_{m,d}$, under axial compression shall be calculated from Equation (13.7-4):

$$U_{m,d} = \frac{\sigma_{c,d}}{f_{c,d} / \gamma_{R,c,d}} \tag{13.7-4}$$

13.7.2.3.2 Column buckling

The representative axial compressive strength for the dented members considered in 13.7.2.3.1 shall be determined from the following equations:

$$f_{c,d} = f_{c,d,o} \qquad \text{for } \Delta y / L \leqslant 0{,}001 \tag{13.7-5}$$

$$\frac{f_{c,d}}{f_{c,d,o}} + \frac{f_{c,d}\ A_d (\Delta y - 0{,}001 L)}{\left(1 - \dfrac{f_{c,d}}{\xi_c\ f_{e,d}}\right) \xi_m\ f_b\ Z_e} = 1.0 \qquad \text{for } \Delta y / L > 0{,}001 \tag{13.7-6}$$

where

Δy is the maximum out-of-straightness of the dented member;

L is the unbraced member length, in the plane of buckling which coincides with the plane of Δy;

$f_{c,d}$ is the representative axial compressive strength of dented members, in stress units;

$f_{c,d,o}$ is the representative axial compressive strength of dented members when $\Delta y / L \leqslant 0{,}001$, in stress units

$$f_{c,d,o} = \left(1{,}0 - 0{,}278\ \lambda_d^2\right) \xi_c\ f_{yc} \qquad \text{for } \lambda_d \leqslant 1{,}34 \tag{13.7-7}$$

$$f_{c,d,o} = \frac{0{,}9}{\lambda_d^2} \xi_c\ f_{yc} \qquad \text{for } \lambda_d > 1{,}34 \tag{13.7-8}$$

f_{yc} is the representative local buckling strength as defined in 13.2.3.3, in stress units;

f_b is the representative bending strength as defined in 13.2.4, in stress units;

$f_{e,d}$ is the Euler buckling strength of the dented member, in stress units

$$f_{e,d} = \frac{\pi^2 E}{\left(K_d L / r_d\right)^2} \tag{13.7-9}$$

Z_e is the elastic section modulus of the undamaged member as defined in 13.2.4;

λ_d is the slenderness parameter of the dented member

$$\lambda_d = \sqrt{\frac{f_{yc}}{f_{e,d}}} = \frac{K_d L}{\pi\, r_d}\sqrt{\frac{f_{yc}}{E}} \quad (13.7\text{-}10)$$

K_d is the effective length factor of the dented member, which may be assumed to be the same as that for the undamaged member as defined in 13.2.3.2;

r_d is the radius of gyration of the dented member, which shall be calculated as

$$r_d = \sqrt{\frac{I_d}{A_d}} \quad (13.7\text{-}11)$$

I_d is the effective moment of inertia of the dented cross-section

$$I_d = \xi_m I_o \quad (13.7\text{-}12)$$

I_o is the moment of inertia of the undamaged member, as defined in 13.2.3.2;

A_d is the effective cross-sectional area of the dented section

$$A_d = \xi_c A \quad (13.7\text{-}13)$$

where

A is the cross-sectional area of the undamaged section, as defined in 13.2.3.2;

ξ_c and ξ_m are coefficients defined by

$$\xi_c = e^{-0{,}08\, h/t} \qquad \text{for } \frac{h}{t} \leqslant 10{,}0 \quad (13.7\text{-}14)$$

$$\xi_m = e^{-0{,}06\, h/t} \qquad \text{for } \frac{h}{t} \leqslant 10{,}0 \quad (13.7\text{-}15)$$

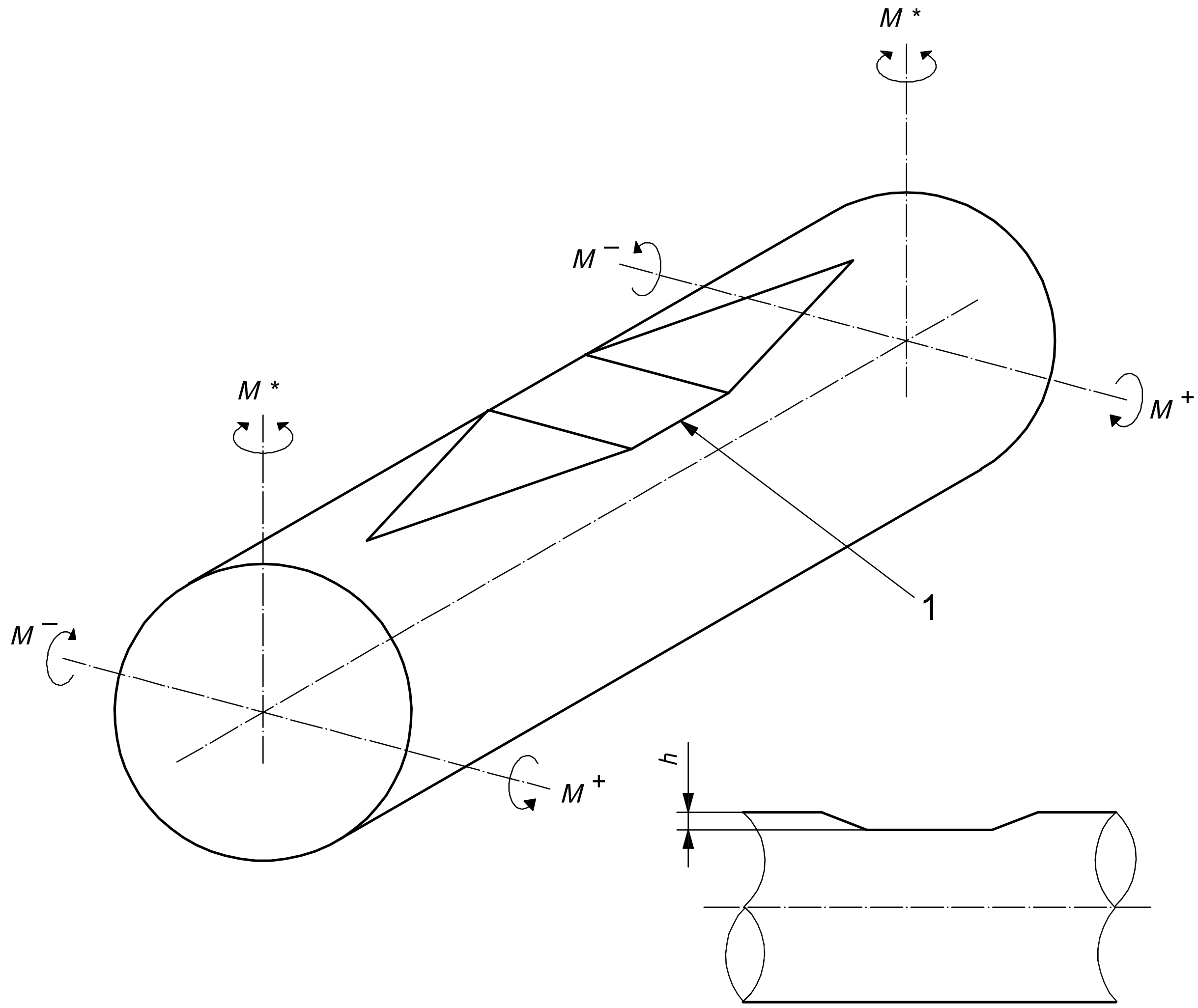
h is the maximum depth of the dent, see Figure 13.7-1;

t is the thickness of the member.

13.7.2.4 Bending

13.7.2.4.1 General

Depending upon the plane of bending, dented cylindrical members subjected to bending can experience tensile, compressive, or zero (neutral) stress conditions existing in the central region of the dent (see Figure 13.7-1). Accordingly, they shall satisfy the requirements of 13.7.2.4.2 to 13.7.2.4.4.



Key

1 dented section of the member

M^+ positive moment — dent in tension

M^- negative moment — dent in compression

M^* neutral moment — zero stress in centre of dent

h maximum depth of the dent

Figure 13.7-1 — Sign convention for dented tubular

13.7.2.4.2 Positive bending

When the dented region is subjected to a positive bending moment M^+:

$$\sigma_{\mathrm{b,d}}^{+} = \frac{M^{+}}{Z_{\mathrm{e}}} \leqslant \frac{f_{\mathrm{b}}}{\gamma_{\mathrm{R,b,d}}} \tag{13.7-16}$$

where

f_{b} is the representative bending strength defined in 13.2.4, in stress units;

$\sigma_{\mathrm{b,d}}^{+}$ is the positive bending stress due to forces from factored actions with respect to the undamaged cross-section, $\sigma_{\mathrm{b,d}}^{+} = M^{+}/Z_{\mathrm{e}}$;

$\gamma_{\mathrm{R,b,d}}$ is the partial resistance factor for bending strength for dented members, $\gamma_{\mathrm{R,b,d}} = 1{,}05$.

The utilization of a dented member, $U_{\text{m,d}}$, under positive bending shall be calculated from Equation (13.7-17):

$$U_{\text{m,d}} = \frac{\sigma^{+}_{\text{b,d}}}{f_{\text{b}} / \gamma_{\text{R,b,d}}} = \frac{M^{+} / Z_{\text{e}}}{f_{\text{b}} / \gamma_{\text{R,b,d}}} \tag{13.7-17}$$

13.7.2.4.3 Negative bending

When the dented region is subjected to a negative bending moment M^{-}:

$$\sigma^{-}_{\text{b,d}} = \frac{M^{-}}{Z_{\text{e}}} \leqslant \xi_{\text{m}} \frac{f_{\text{b}}}{\gamma_{\text{R,b,d}}} \tag{13.7-18}$$

where $\sigma^{-}_{\text{b,d}}$ is the negative bending stress due to forces from factored actions with respect to the undamaged cross-section, $\sigma^{-}_{\text{b,d}} = M^{-}/Z_{\text{e}}$.

The utilization of a dented member, $U_{\text{m,d}}$, under negative bending shall be calculated from Equation (13.7-19):

$$U_{\text{m,d}} = \frac{\sigma^{-}_{\text{b,d}}}{\xi_{\text{m}}\, f_{\text{b}} / \gamma_{\text{R,b,d}}} = \frac{M^{-} / Z_{\text{e}}}{\xi_{\text{m}}\, f_{\text{b}} / \gamma_{\text{R,b,d}}} \tag{13.7-19}$$

where ξ_{m} is as defined in 13.7.2.3.2.

13.7.2.4.4 Neutral bending

When the dented region is subjected to a neutral bending moment M^{*}:

$$\sigma^{*}_{\text{b,d}} = \frac{M^{*}}{Z_{\text{e}}} \leqslant \frac{f_{\text{b}}}{\gamma_{\text{R,b,d}}} \tag{13.7-20}$$

where $\sigma^{*}_{\text{b,d}}$ is the neutral bending stress due to forces from factored actions with respect to the undamaged cross-section, $\sigma^{*}_{\text{b,d}} = M^{*}/Z_{\text{e}}$.

The utilization of a dented member, $U_{\text{m,d}}$, under neutral bending shall be calculated from Equation (13.7-21):

$$U_{\text{m,d}} = \frac{\sigma^{*}_{\text{b,d}}}{f_{\text{b}} / \gamma_{\text{R,b,d}}} = \frac{M^{*} / Z_{\text{e}}}{f_{\text{b}} / \gamma_{\text{R,b,d}}} \tag{13.7-21}$$

13.7.2.5 Shear

Dented tubular members subjected to beam shear forces shall satisfy the condition specified in 13.2.5.1 in which the maximum shear stress due to forces from factored actions should be approximated by the following equations:

$$\tau_{\text{b,d}} = \frac{2V}{A} \qquad \text{for } h \leqslant 0{,}25D \tag{13.7-22}$$

$$\tau_{\text{b,d}} = \frac{2V/A}{(1{,}5 - 2h/D)} \qquad \text{for } 0{,}25D < h \leqslant 0{,}3D \tag{13.7-23}$$

where V and A are as defined in 13.2.5.1.

13.7.3 Dented tubular members subjected to combined forces

13.7.3.1 Axial tension and bending

13.7.3.1.1 General

Dented tubular members subjected to combined axial tension and bending forces shall satisfy the following conditions at all cross-sections along their length.

13.7.3.1.2 Axial tension, positive bending and neutral bending

Dented tubular members shall satisfy Equation (13.7-24):

$$\frac{\gamma_{R,t,d}\ \sigma_{t,d}}{f_y} + \frac{\gamma_{R,b,d}\sqrt{\left(\overset{+}{\sigma}_{b,d}\right)^2 + \left(\overset{*}{\sigma}_{b,d}\right)^2}}{f_b} \leqslant 1{,}0 \tag{13.7-24}$$

The utilization of a dented member, $U_{m,d}$, under axial tension, positive bending and neutral bending shall be calculated using Equation (13.7-25):

$$U_{m,d} = \frac{\gamma_{R,t,d}\ \sigma_{t,d}}{f_y} + \frac{\gamma_{R,b,d}\sqrt{\left(\overset{+}{\sigma}_{b,d}\right)^2 + \left(\overset{*}{\sigma}_{b,d}\right)^2}}{f_b} \tag{13.7-25}$$

13.7.3.1.3 Axial tension, negative bending and neutral bending

Dented tubular members shall satisfy Equation (13.7-26):

$$\frac{\gamma_{R,t,d}\ \sigma_{t,d}}{f_y} + \sqrt{\left(\frac{\gamma_{R,b,d}\ \overset{-}{\sigma}_{b,d}}{\xi_m\ f_b}\right)^{\alpha} + \left(\frac{\gamma_{R,b,d}\ \overset{*}{\sigma}_{b,d}}{f_b}\right)^2} \leqslant 1{,}0 \tag{13.7-26}$$

where

$$\alpha = 2 - 3\frac{h}{D} \tag{13.7-27}$$

and ξ_m is as defined in 13.7.2.3.2

The utilization of a dented member, $U_{m,d}$, under axial tension, negative bending and neutral bending shall be calculated from Equation (13.7-28):

$$U_{m,d} = \frac{\gamma_{R,t,d}\ \sigma_{t,d}}{f_y} + \sqrt{\left(\frac{\gamma_{R,b,d}\ \overset{-}{\sigma}_{b,d}}{\xi_m\ f_b}\right)^{\alpha} + \left(\frac{\gamma_{R,b,d}\ \overset{*}{\sigma}_{b,d}}{f_b}\right)^2} \tag{13.7-28}$$

13.7.3.2 Axial compression and bending

13.7.3.2.1 General

Dented tubular members subjected to combined axial compressive and bending forces shall satisfy the following conditions at all sections along their length.

13.7.3.2.2 Axial compression, positive bending and neutral bending

Dented tubular members shall satisfy Equations (13.7-29) and (13.7-30):

$$\frac{\gamma_{R,c,d}\ \sigma_{c,d}}{f_{c,d}}+\frac{\gamma_{R,b,d}}{f_b}\left[\left(\frac{\overset{+}{\sigma}_{b,d}}{\left(1-\sigma_{c,d}/f_e\right)}\right)^2+\left(\frac{\overset{*}{\sigma}_{b,d}}{\left(1-\sigma_{c,d}/f_e\right)}\right)^2\right]^{0,5}\leqslant 1{,}0 \tag{13.7-29}$$

$$\frac{\gamma_{R,c,d}\ \sigma_{c,d}}{f_{yc,d}}+\frac{\gamma_{R,b,d}\sqrt{\left(\overset{+}{\sigma}_{b,d}\right)^2+\left(\overset{*}{\sigma}_{b,d}\right)^2}}{f_b}\leqslant 1{,}0 \tag{13.7-30}$$

where

$f_{yc,d}$ is the representative local buckling strength of the dented member, in stress units;

$$f_{yc,d}=\xi_c f_{yc} \tag{13.7-31}$$

ξ_c is as defined in 13.7.2.3.2;

f_{yc} is the representative local buckling strength of the undamaged member, as defined in 13.2.3.3;

f_e is the smaller of the Euler buckling strengths of the undamaged member in the positive and neutral bending directions, in stress units, as defined in 13.3.3;

The utilization of a dented member, $U_{m,d}$, under axial compression, positive bending and neutral bending, shall be the larger value calculated from Equations (13.7-32) and (13.7-33):

$$U_{m,d}=\frac{\gamma_{R,c,d}\ \sigma_{c,d}}{f_{c,d}}+\frac{\gamma_{R,b,d}}{f_b}\left[\left(\frac{\overset{+}{\sigma}_{b,d}}{\left(1-\sigma_{c,d}/f_e\right)}\right)^2+\left(\frac{\overset{*}{\sigma}_{b,d}}{\left(1-\sigma_{c,d}/f_e\right)}\right)^2\right]^{0,5} \tag{13.7-32}$$

$$U_{m,d}=\frac{\gamma_{R,c,d}\ \sigma_{c,d}}{f_{yc,d}}+\frac{\gamma_{R,b,d}\sqrt{\left(\overset{+}{\sigma}_{b,d}\right)^2+\left(\overset{*}{\sigma}_{b,d}\right)^2}}{f_b} \tag{13.7-33}$$

13.7.3.2.3 Axial compression, negative bending and neutral bending

Dented tubular members shall satisfy Equations (13.7-34) and (13.7-35):

$$\frac{\gamma_{R,c,d}\ \sigma_{c,d}}{f_{c,d}}+\left\{\left[\frac{\gamma_{R,b,d}\ \overset{-}{\sigma}_{b,d}}{\left(1-\dfrac{\sigma_{c,d}}{\xi_c\ f_{e,d}}\right)\xi_m\ f_b}\right]^{\alpha}+\left[\frac{\gamma_{R,b,d}\ \overset{*}{\sigma}_{b,d}}{\left(1-\dfrac{\sigma_{c,d}}{f_e}\right)f_b}\right]^2\right\}^{0,5}\leqslant 1{,}0 \tag{13.7-34}$$

$$\frac{\gamma_{R,c,d}\ \sigma_{c,d}}{f_{yc,d}}+\left[\left(\frac{\gamma_{R,b,d}\ \overset{-}{\sigma}_{b,d}}{\xi_m\ f_b}\right)^{\alpha}+\left(\frac{\gamma_{R,b,d}\ \overset{*}{\sigma}_{b,d}}{f_b}\right)^2\right]^{0,5}\leqslant 1{,}0 \tag{13.7-35}$$

where

α is as defined in 13.7.3.1.3;

$f_{e,d}$ is as defined in 13.7.2.3.2;

f_e is the Euler buckling strength of the undamaged member in the neutral bending direction as defined in 13.3.3;

The utilization of a dented member, $U_{m,d}$, under axial compression, negative bending and neutral bending shall be the larger value calculated from Equations (13.7-36) and (13.7-37):

$$U_{m,d} = \frac{\gamma_{R,c,d}\ \sigma_{c,d}}{f_{c,d}} + \left\{ \left[\frac{\gamma_{R,b,d}\ \bar{\sigma}_{b,d}}{\left(1 - \dfrac{\sigma_{c,d}}{\xi_c f_{e,d}}\right) \xi_m\ f_b} \right]^{\alpha} + \left[\frac{\gamma_{R,b,d}\ \sigma^*_{b,d}}{\left(1 - \dfrac{\sigma_{c,d}}{f_e}\right) f_b} \right]^2 \right\}^{0,5} \quad (13.7\text{-}36)$$

$$U_{m,d} = \frac{\gamma_{R,c,d}\ \sigma_{c,d}}{f_{yc,d}} + \left[\left(\frac{\gamma_{R,b,d}\ \bar{\sigma}_{b,d}}{\xi_m\ f_b} \right)^{\alpha} + \left(\frac{\gamma_{R,b,d}\ \sigma^*_{b,d}}{f_b} \right)^2 \right]^{0,5} \quad (13.7\text{-}37)$$

13.8 Corroded tubular members

The presence of local or overall corrosion should be taken into account in determining the residual strength of a corroded tubular member. At this time there is insufficient information on the strength of corroded tubular members for the inclusion of requirements in this International Standard. However, a methodology for estimating the effect of corrosion on member strength is discussed in A.13.8.

13.9 Grouted tubular members

13.9.1 General

Subclause 13.9 applies to both fully grouted undamaged members and fully grouted dented tubular members, with the dent depth h limited to either $h \leqslant 0{,}3\ D$ or $h \leqslant 10\ t$. For grouted tubular members, neither the effect of external hydrostatic pressure nor the presence of cracks has been taken into account in the derivation of the equations given in this subclause. Such effects shall be considered on a case-by-case basis.

Requirements for partially grouted tubular members are excluded, owing to the limited availability of relevant test data (see A.13.9.1 for additional information).

The strength model recognizes that grout filling inhibits the development of local buckling of undamaged tubulars and the growth of dents in damaged tubulars. Therefore, it is assumed that completely grouted tubular members can undergo column buckling, but not local buckling. However, in cases where local buckling can be important (e.g. large D/t), the representative steel yield strength, f_y, may be replaced by the representative local buckling strength, f_{yc} (see 13.2.3.3).

13.9.2 Grouted tubular members subjected to tension, compression or bending

13.9.2.1 General

Grouted tubular members subjected independently to axial tension, axial compression or bending shall be assessed using the strength and stability requirements specified in 13.9.2.2 to 13.9.2.4.

13.9.2.2 Axial tension

Fully grouted tubular members subjected to axial tensile forces shall normally satisfy the following condition:

$$\sigma_{t,g} \leqslant \frac{f_t}{\gamma_{R,t,g}} \qquad (13.9\text{-}1)$$

where

$\sigma_{t,g}$ is the axial tensile stress in the steel tubular member due to forces from factored actions, neglecting the grout;

f_t is the representative axial tensile strength of the steel, in stress units, $f_t = f_y$;

f_y is the representative yield strength of the steel, in stress units;

$\gamma_{R,t,g}$ is the partial resistance factor for axial tensile strength of the grouted member, $\gamma_{R,t,g}$ = 1,05.

Where it can be demonstrated that complete grouting of the tubular has been achieved, f_t may be taken as 1,12 f_y.

The utilization of a grouted tubular member, $U_{m,g}$, under axial tension shall be calculated from Equation (13.9-2):

$$U_{m,g} = \frac{\sigma_{t,g}}{f_t / \gamma_{R,t,g}} \qquad (13.9\text{-}2)$$

13.9.2.3 Axial compression

Fully grouted tubular members subjected to axial compressive forces shall satisfy the following condition:

$$\sigma_{c,g} \leqslant \frac{f_{c,g}}{\gamma_{R,c,g}} \qquad (13.9\text{-}3)$$

where

$\sigma_{c,g}$ is the axial compressive stress in the fully grouted member due to forces from factored actions acting on the transformed area, $\sigma_{c,g} = P/A_{tr}$;

$f_{c,g}$ is the representative axial compressive strength of the grouted member, in stress units, as defined in Equations (13.9-8) and (13.9-9);

$\gamma_{R,c,g}$ is the partial resistance factor for axial compressive strength of the grouted member, $\gamma_{R,c,g}$ = 1,18;

P is the axial compressive force in the grouted member due to factored actions;

A_{tr} is the transformed area of the fully grouted member

$$A_{tr} = A_s + \frac{A_g}{m} \qquad (13.9\text{-}4)$$

A_s is the cross-sectional area of the steel

$$A_s = (D-t)t\left[\pi - \left(\alpha_g - \sin\alpha_g\right)\right] \qquad (13.9\text{-}5)$$

$$\alpha_g = \frac{1}{\cos\left[1-2h/(D-t)\right]} \qquad (13.9\text{-}6)$$

A_g is the cross-sectional area of the grout

$$A_g = (D - 2t)^2 \left(\pi - \alpha_g + 1/2 \sin 2\alpha_g \right) / 4 \tag{13.9-7}$$

m is the ratio of elastic moduli of steel and grout, $m = E_s/E_g$ (m = 18, in lieu of actual data);

E_s is Young's modulus of elasticity for steel;

E_g is the modulus of elasticity of grout;

h is the maximum dent depth, if present;

D is the outer diameter of the steel tubular member;

t is the thickness of the steel.

The representative axial compressive strength of fully grouted tubular members is due to column buckling and shall be determined from the following equations:

$$f_{c,g} = \left(1{,}0 - 0{,}278 \lambda_g^{\ 2} \right) f_{ug} \qquad \text{for } \lambda_g \leqslant 1{,}34 \tag{13.9-8}$$

$$f_{c,g} = \frac{0{,}9}{\lambda_g^{\ 2}} f_{ug} \qquad \text{for } \lambda_g > 1{,}34 \tag{13.9-9}$$

$$\lambda_g = \sqrt{\frac{f_{ug}}{f_{e,g}}} \tag{13.9-10}$$

where, in addition to the definitions given above,

λ_g is the column slenderness parameter of the grouted member;

f_{ug} is the axial squash strength of the grouted member, in stress units

$$f_{ug} = (A_s f_y + 0{,}67 A_g f_{cu}) / A_{tr} \tag{13.9-11}$$

f_y is the representative yield strength of the steel, in stress units;

f_{cu} is the representative unconfined cube strength of grout, in stress units;

$f_{e,\,g}$ is the Euler buckling strength of the fully grouted member, in stress units

$$f_{e,g} = \frac{\pi^2 \left(E_s I_s + 0{,}8 E_g I_g \right)}{A_{tr} \left(KL \right)^2} \tag{13.9-12}$$

K is the effective length factor (see 13.5);

L is the longer of the unbraced lengths in the y- and z-directions;

I_s is the effective moment of inertia of the steel cross-section

$$I_s = \left[(D-t)^3 \, t \left(\pi - \alpha_g - \tfrac{1}{2} \sin 2\alpha_g + 2 \sin \alpha_g \cos^2 \alpha_g \right) \Big/ 8 \right] - A_s \, e_s^2 \tag{13.9-13}$$

I_g is the effective moment of inertia of the grout cross-section

$$I_g = \left[(D-2t)^4 \left(\pi - \alpha_g + \tfrac{1}{4} \sin 4\alpha_g \right) \Big/ 64 \right] - A_g e_g^2 \tag{13.9-14}$$

$$e_s = \tfrac{1}{2} (D-t)^2 \, t \sin \alpha_g \left(1 - \cos \alpha_g \right) / A_s \tag{13.9-15}$$

$$e_g = (D-2t)^3 \sin^3 \alpha_g \Big/ \left(12 A_g \right) \tag{13.9-16}$$

The utilization of a grouted member, $U_{m,g}$, under axial compression shall be calculated from Equation (13.9-17):

$$U_{m,g} = \frac{\sigma_{c,g}}{f_{c,g} / \gamma_{R,c,g}} \tag{13.9-17}$$

13.9.2.4 Bending

Fully grouted tubular members subjected to bending forces shall satisfy the following condition:

$$\sigma_{b,g} = \frac{M}{Z_e} \leqslant \frac{f_{b,g}}{\gamma_{R,b,g}} \tag{13.9-18}$$

where, in addition to the definitions given in 13.2.4, 13.9.2.2 and 13.9.2.3,

$\sigma_{b,g}$ is the bending stress due to forces from factored actions and, when $\sigma_{b,g} > f_{b,g}$, is to be considered as an equivalent elastic bending stress, $\sigma_{b,g} = M/Z_e$;

M is the bending moment in the grouted member due to factored actions;

$f_{b,g}$ is the representative bending strength of the grouted member, in stress units, defined as

$$f_{b,g} = \frac{Z_p}{Z_e} f_y \, \delta \left(1 + 0{,}01 k \right) \tag{13.9-19}$$

$$\delta = 1 - 0{,}5 \frac{h}{D} - 1{,}6 \left(\frac{h}{D} \right)^2 \tag{13.9-20}$$

$$k = 5{,}5 \delta \left(\rho \frac{D}{t} \right)^{0{,}66} \tag{13.9-21}$$

$$\rho = \frac{0{,}6 f_{cu}}{f_y} \tag{13.9-22}$$

Z_e, Z_p are calculated as in 13.2.4 for undamaged and ungrouted members;

$\gamma_{R,b,g}$ is the partial resistance factor for the bending strength of grouted members, $\gamma_{R,b,g} = 1{,}05$.

The utilization of a grouted tubular member, $U_{m,g}$, under bending shall be calculated from Equation (13.9-23):

$$U_{m,g} = \frac{\sigma_{b,g}}{f_{b,g} / \gamma_{R,b,g}} = \frac{M / Z_e}{f_{b,g} / \gamma_{R,b,g}} \tag{13.9-23}$$

13.9.3 Grouted tubular members subjected to combined forces

13.9.3.1 Axial tension and bending

No requirements for fully grouted tubular members subjected to axial tension and bending forces are presented because of the limited availability of relevant test data. However, several approximate methods that may be used to estimate the interaction are discussed in A.13.9.3.1.

13.9.3.2 Axial compression and bending

Fully grouted tubular members subjected to combined axial compressive and bending forces shall satisfy the following conditions at all cross-sections along their length for undamaged tubulars, and along the dented section for damaged tubulars:

$$\frac{\gamma_{R,c,g}\,\sigma_{c,g}}{f_{c,g}} + \frac{\gamma_{R,b,g}\,T_1\,\sigma_{b,g}}{f_{b,g}} + \frac{T_2\left(\gamma_{R,b,g}\,\sigma_{b,g}\right)^2}{{f_{b,g}}^2} \leqslant 1{,}0 \quad \text{for} \quad \frac{\gamma_{R,c,g}\,\sigma_{c,g}}{f_{c,g}} \geqslant \frac{K_2}{K_1} \tag{13.9-24}$$

$$\frac{\gamma_{R,b,g}\,\sigma_{b,g}}{f_{b,g}} \leqslant 1{,}0 \quad \text{for} \quad \frac{\gamma_{R,c,g}\,\sigma_{c,g}}{f_{c,g}} < \frac{K_2}{K_1} \tag{13.9-25}$$

where, in addition to the definitions given in 13.9.2.3 and 13.9.2.4,

$$T_1 = 1 - \frac{K_2}{K_1} - T_2 \tag{13.9-26}$$

$$T_2 = 4\frac{K_3}{K_1} \tag{13.9-27}$$

$$K_1 = 1{,}0 - 0{,}278\,\lambda_g^2 \quad \text{for} \quad \lambda_g \leqslant 1{,}34 \tag{13.9-28}$$

$$K_1 = 0{,}9 \big/ \lambda_g^2 \quad \text{for} \quad \lambda_g > 1{,}34 \tag{13.9-29}$$

$$K_2 = \frac{K_{20}\left[115 - 30(2\beta - 1)(1{,}8 - \theta) - 100\lambda_g\right]}{50(2{,}1 - \beta)} \quad \text{for} \quad 0 \leqslant K_2 \leqslant K_{20} \tag{13.9-30}$$

$$K_3 = K_{30} + \frac{\lambda_g\left[(0{,}5\beta + 0{,}4)\left(\theta^2 - 0{,}5\right) + 0{,}15\right]}{1 + \lambda_g^3} \tag{13.9-31}$$

$$K_{20} = \left(0{,}9\theta^2 + 0{,}2\right) \leqslant 0{,}75 \tag{13.9-32}$$

$$K_{30} = \left(0{,}04 - \theta/15\right) \geqslant 0 \tag{13.9-33}$$

$$\theta = \frac{0{,}67 A_{\mathrm{g}} \left(f_{\mathrm{cu}} + \dfrac{C_1 f_{\mathrm{y}} t}{D} \right)}{f_{\mathrm{ug}} A_{\mathrm{tr}}} \tag{13.9-34}$$

β is the ratio of the smaller to the larger end moment, with $\beta = 1$ if no end moments apply;

$$C_1 = 4\phi\varepsilon\left(1 + \phi + \phi^2\right)^{-0{,}5} \tag{13.9-35}$$

$$\phi = 0{,}02\left(25 - \frac{K L}{D}\right) \geqslant 0 \tag{13.9-36}$$

$$\varepsilon = 0{,}25\left(25 - \frac{K L}{D}\right) \geqslant 0 \tag{13.9-37}$$

The utilization of a grouted tubular member, $U_{\mathrm{m,g}}$, under axial compression and bending shall be the larger value calculated from Equations (13.9-38) and (13.9-39):

$$U_{\mathrm{m,g}} = \frac{\gamma_{\mathrm{R,c,g}}\, \sigma_{\mathrm{c,g}}}{f_{\mathrm{c,g}}} + \frac{\gamma_{\mathrm{R,b,g}}\, T_1\, \sigma_{\mathrm{b,g}}}{f_{\mathrm{b,g}}} + \frac{T_2 \left(\gamma_{\mathrm{R,b,g}}\, \sigma_{\mathrm{b,g}}\right)^2}{f_{\mathrm{b,g}}^{\;2}} \tag{13.9-38}$$

$$U_{\mathrm{m,g}} = \frac{\gamma_{\mathrm{R,b,g}}\, \sigma_{\mathrm{b,g}}}{f_{\mathrm{b,g}}} \tag{13.9-39}$$

14 Strength of tubular joints

14.1 General

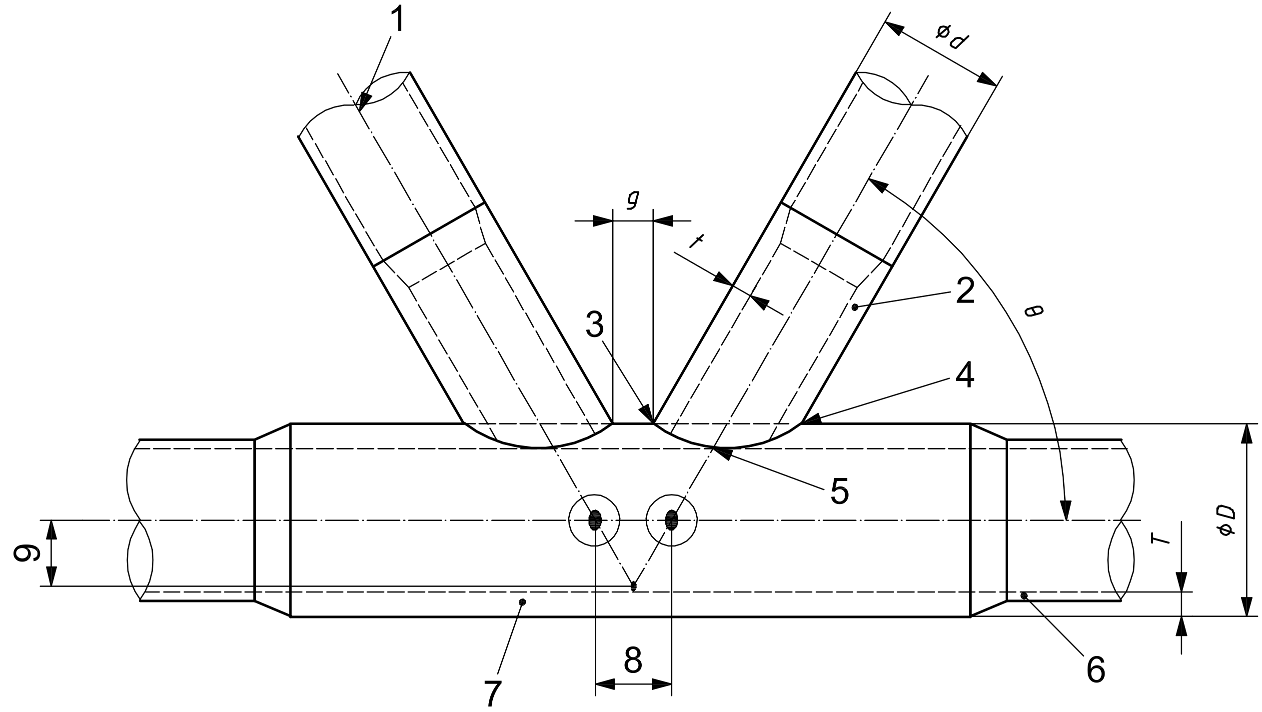
The requirements given in this clause apply to the static design of tubular joints formed by the connection of two or more members. Generic requirements for non-tubular joints are also given. Joint types are classified in 14.2.4.

In lieu of the requirements in this International Standard, reasonable alternative methods may be used for the design of joints. Test data and analytical techniques may be used as a basis for design, provided that it can be demonstrated that the strength of such joints can be determined reliably. Such analytical or numerical techniques should always be calibrated and benchmarked to suitable test data.

The requirements have been derived from a consideration of the representative strength (as opposed to the mean strength) of tubular joints. Representative strength is comparable to lower bound strength. The background to, and a discussion of, the requirements are presented in A.14.

Care should be taken in using the results of very limited test programmes or analytical investigations to provide an estimate of joint strength, since very limited test programmes form an improper basis for determining the representative value (see 7.7). Consideration shall, in such cases, be given to the imposition of a reduction factor on the calculation of joint strength, in order to account for the small amount of data or for a poor basis of the calculations.

The nomenclature for simple joints is given in Figure 14.1-1.



Key

1	brace	θ	included angle between chord and brace axes
2	stub (where present)	g	gap between braces, negative for overlapped stubs
3	crown toe	t	brace wall thickness at intersection
4	crown heel	T	chord wall thickness at intersection
5	saddle	d	brace outside diameter
6	chord	D	chord outside diameter
7	can		
8	offset		
9	eccentricity		

$$\beta = \frac{d}{D} \qquad \gamma = \frac{D}{2T} \qquad \tau = \frac{t}{T}$$

Figure 14.1-1 — Terminology and geometrical parameters for simple tubular joints

14.2 Design considerations

14.2.1 Materials

The general requirements for materials are given in Clause 19, while additional requirements specific to the strength of tubular connections are given below.

The representative yield strength of the steel shall be taken as the specified minimum yield strength (SMYS), except that for chord materials with a SMYS of 500 MPa or less, the representative yield strength shall not exceed 80 % of the tensile strength. A.14.2.1 gives additional information on materials with a minimum specified yield strength greater than 500 MPa.

Welds in fabricated joints shall be designed to develop a strength greater than or equal to both the yield strength of the nominal brace cross-section (ignoring any brace stubs) and the full strength of the joint. Further guidelines for welds for circular tubular joints are given in Clause 20.

Joints often involve welds from several brace connections in close proximity. The high restraint of joints can cause large strain concentrations and a potential for cracking or lamellar tearing. Hence the chord material (and brace/stub material, if overlapping is present) shall have adequate through-thickness toughness, see Clause 19.

There is sometimes uncertainty in the material properties in structures that are being assessed (see Clause 24) or reused (see Clause 25). In these instances, testing of material removed from the actual structure can be required. If the through-thickness toughness of joint can steel cannot be determined, inspection for possible cracks or lamellar tearing shall be considered.

Recommendations for grout materials for use in grouted joints are given in 19.6.

14.2.2 Design forces and joint flexibility

Joints shall be designed and assessed using internal forces resulting from factored actions in accordance with Clauses 8 to 11. In addition, for the design of new structures, joints for all primary structural members shall be at least as strong as the adjoining braces, see 14.2.3.

The reduction in secondary (deflection induced) bending moments due to joint flexibility or due to inelastic relaxation may be considered. For ultimate strength analysis of the structure, information on the force-deformation characteristics of joints may be used. These characteristics are dependent on the joint type, configuration, geometry, material properties, load case under consideration and, in certain instances, hydrostatic pressure effects; see A.14.2.2 for a further discussion on joint flexibility.

14.2.3 Minimum strength

The requirement for the strength of joints is given in general form in Equation (14.2-1):

$$S_{\mathrm{j}} \leqslant \frac{R_{\mathrm{j}}}{\gamma_{\mathrm{R,j}}} \tag{14.2-1}$$

where

S_{j} is the generalized internal force in the joint;

R_{j} is the corresponding generalized resistance of the joint;

$\gamma_{\mathrm{R,j}}$ is the partial resistance factor for joints, $\gamma_{\mathrm{R,j}} = 1{,}05$.

All joints, except those identified as being non-critical, shall additionally be checked to ensure that joint strength exceeds the brace member strength, using Equation (14.2-2):

$$\frac{\gamma_{\mathrm{R,j}}\, S_{\mathrm{j}}}{R_{\mathrm{j}}} \leqslant \frac{U_{\mathrm{b}}}{\gamma_{\mathrm{zj}}} \tag{14.2-2}$$

where, additionally,

U_{b} is the utilization of the brace (see Clause 13) at the end adjoining the joint, which may conservatively be taken as the maximum utilization along the brace or even more conservatively as unity;

γ_{zj} is an extra partial resistance factor to ensure that members fail before the joint yields.

The total resistance factor for joint strength in relation to brace utilization is the product of γ_{R} and γ_{zj}. γ_{zj} shall normally be taken as 1,17, giving a total resistance factor of 1,23; γ_{zj} may be relaxed to a value within the range 1,00 to 1,17 only if this can be justified by the designer, giving a total resistance factor between 1,05 and 1,23.

Non-critical joints are joints that do not

— influence the reserve strength of a structure,

— influence the response of a structure when subjected to accidental events, or

— cause significant safety or environmental consequences if they fail.

In practice, the checks in Equations (14.2-1) and (14.2-2) for combined forces and moments consider the interaction between the forces, the moments, the resistances to forces and the resistances to moments. The strength of a simple tubular joint shall be checked using the interaction equation, see Equation (14.3-12). Interaction for simple tubular joints shall be checked using Equation (14.3-13), except for those joints identified as being non-critical.

14.2.4 Joint classification

There are three basic planar joint types, these being Y-, K- and X-joints, as shown in Figure 14.2-1 and as described below.

— A Y-joint consists of a chord and one brace. Axial force in the brace is reacted by an axial force and beam shear in the chord.

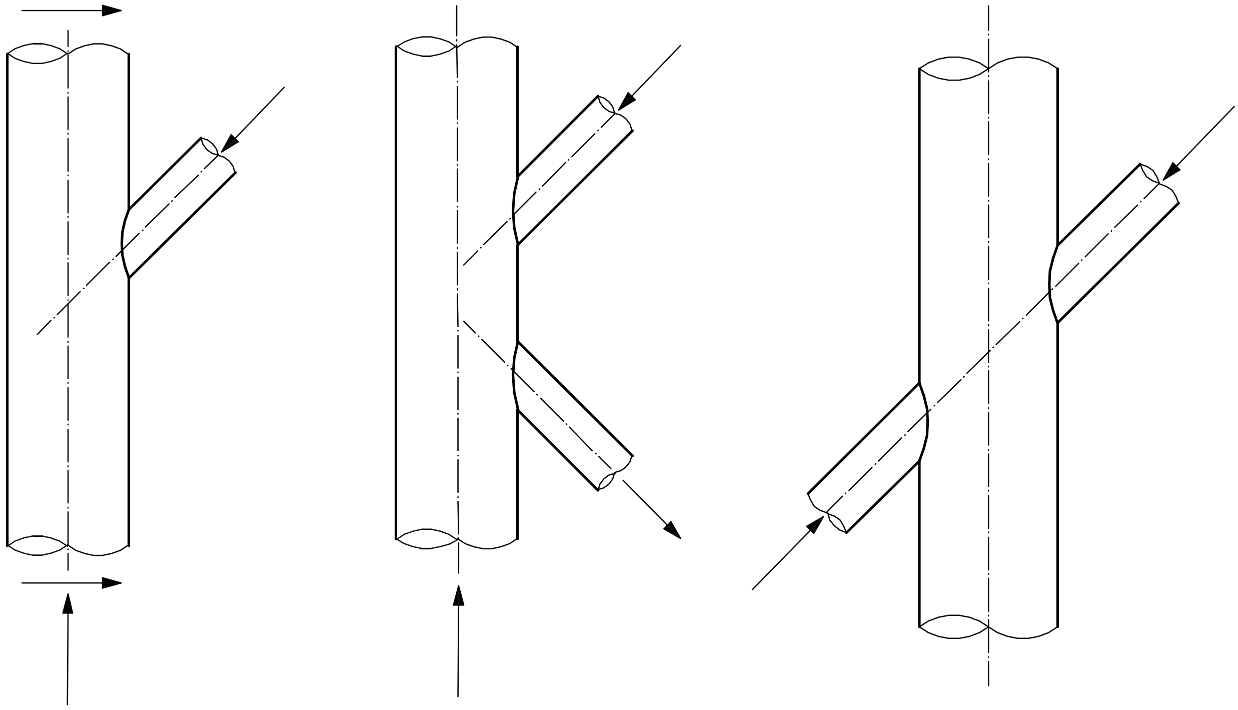
— A K-joint consists of a chord and two braces on the same side of the chord. The components of the axial brace forces normal to the chord balance each other, while the components parallel to the chord add and are reacted by an axial force in the chord.

— An X-joint (also called *cross-joint*) consists of a chord and two braces, one on each side of the chord, where the second brace is a continuation of the first brace. Axial force in one brace is transferred through the chord to the other brace without an overall reaction in the chord.

In all joint types, the chord is the through member.

Many joints are combinations of the above joint types, containing mixtures of behaviour either in one plane or in several planes (multi-planar joints). A T-joint is a Y-joint in which the angle between the brace and the chord is approximately 90°. A DT-joint, or double T-joint, looks like an X-joint with angles of approximately 90° but behaves as two T-joints, in that the axial brace forces are transferred to the chord rather than crossing the chord to the other brace.

Joint classification between Y-, K- and X-joints is based solely on consideration of the axial forces in the braces.



a) Y-joint b) K-joint c) X-joint

Figure 14.2-1 — Basic planar joint types

The design strength of most joints can be determined using the parametric formulae given in 14.3 for the three basic planar joint types. However, fixed steel offshore structures are normally space frames, containing both multiplanar joints and simple Y-, K- and X-joints. The practical use of the basic joint formulae shall reflect, as closely as possible, the force pattern assumed in deriving the formulae by classifying each combination of brace(s) and chord according to the flow of the axial force in the brace(s). A joint should be classified as combinations of Y-, K- and X-joints when the behaviour of the braces contains elements of the behaviour of more than one type. The following approach shall be followed.

Classification as a Y-, K- or X-joint shall apply to the combination of an individual brace with the chord, rather than to the whole joint, on the basis of the axial force pattern for each load case. This classification is relevant to both fatigue and strength considerations.

The classification of each individual brace-chord combination for a given load case shall be as a Y-, K- or X-joint. If the brace-chord combination carries part of the axial brace force as a K-joint, and part as a Y-joint or X-joint, it shall be classified as a proportion of each relevant type, e.g. 50 % as a K-joint and 50 % as an X-joint.

The subdivision in Y-, K- and X-joint axial force patterns normally considers all members in one plane at a joint; brace planes within ±15° of each other may be considered as being in the same plane.

The classification should be based on the following:

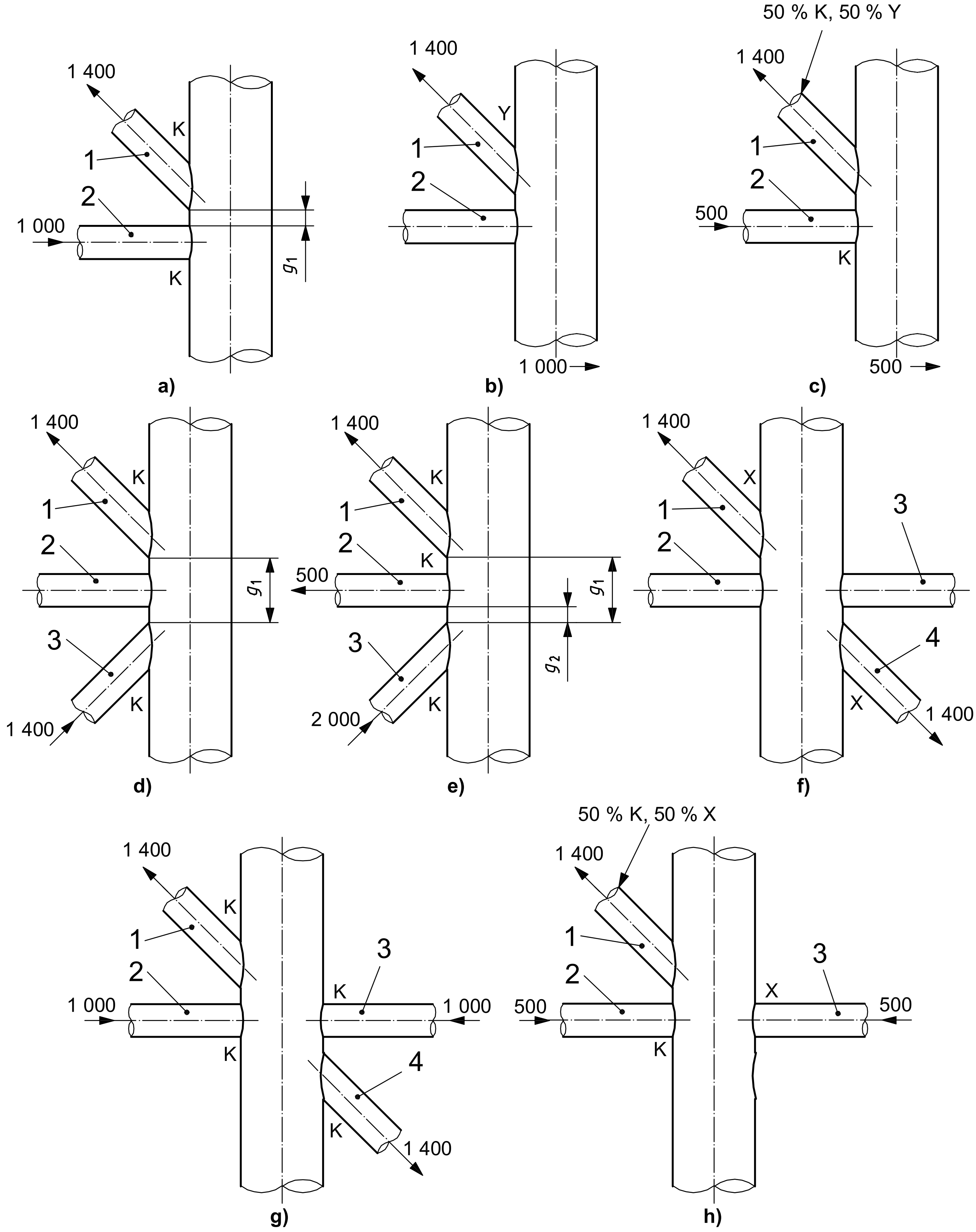
a) a brace should be classified as a K-joint only if the component of axial force in the brace perpendicular to the chord is balanced to within 10 % by force components (perpendicular to the chord) in other braces in the same plane and on the same side of the joint;

b) a brace should be considered as a Y-joint if it does not meet the criteria for a K-joint and if the component of axial force in the brace perpendicular to the chord is reacted as beam shear in the chord;

c) a brace should be considered as an X-joint if it does not meet the criteria for a K-joint or a Y-joint; in this classification the axial force in the brace is transferred through the chord to the opposite side (e.g. to other braces, to padeyes, launch rails or similar structural components).

Figure 14.2-2 shows some simple examples of the brace joint classification scheme.

Alternative classification strategies may be used, such as assigning classification in the order K-, X- and, finally, Y-joint response.



Key

g_1 gap 1

g_2 gap 2

Figure 14.2-2 — Examples of joint classification

Figure 14.2-2 h) is a good example of the axial force flow and classification hierarchy that should be adopted in the classification of braces in joints. The braces 1 and 2 on the left hand side of the chord act as a K-joint accounting for 50 % of the axial force in the diagonal brace. The other 50 % of the axial force in brace 1 forms an X-joint with brace 3. Replacement of brace axial forces by a combination of tension and compression forces to give the same net force is not permitted. For the example shown in Figure 14.2-2 h), replacing the axial force in brace 2 by a compression force of 1 000 and a tension force of 500 is not permitted, as this will result in an inappropriate X-joint classification for this horizontal brace and a full K-joint classification for brace 1.

Careful consideration should be given to determining the correct gap between braces in a K-joint. In Figure 14.2-2 a) the appropriate gap is between adjacent braces. However, if an intermediate brace exists, as in Figure 14.2-2 d), the appropriate gap is between the outer braces acting as the K-joint. In this case, since the gap is often large, the K-joint strength can revert to that of a Y-joint. Figure 14.2-2 e) is instructive in that the appropriate gap for brace 2 is g_2, whereas for brace 1 it is g_1. Although brace 3 is classified wholly as a K-joint (with brace 2 for 500 normal to the chord and with brace 1 for the remainder of the normal component of brace 3), the strength is determined by weighting the strength with gaps of g_1 and g_2 by the proportions of the axial force balancing from braces 1 and 2.

There are some instances where the joint behaviour is more difficult to define or is apparently worse than predicted using the above classification. Two of the more common cases in the latter category are associated with actions on a launch frame and with *in situ* actions on skirt pile-sleeves. Some guidance for such instances is given in A.14.2.4.

Once the breakdown according to axial brace force components is established, the strength of the joint can be determined using the procedures in 14.3.

14.2.5 Detailing practice

Joint detailing is an essential element of joint design. For unreinforced joints, the recommended detailing nomenclature and dimensioning are shown in Figures 14.2-3 and 14.2-4. Where an increased wall thickness or higher yield or toughness properties is required for the chord, this material should extend beyond the outside edge of incoming bracing by the greater of a minimum of one quarter of the chord diameter, or 300 mm. The strength of Y- and X-joints is a function of the can length (see 14.3.5) and short can lengths can lead to a reduction of the joint strength. Increasing the can lengths beyond the minimum values given here should be considered to avoid the need for downgrading strength.

When two or more tubulars join in an X-joint, the larger diameter member shall continue through the joint, and the other should frame onto the through member and be considered the minor member. Where members of equal diameter meet at an X-joint, it is more efficient to make the through member that which carries the greater forces. Unless specified otherwise on the drawings, when two or more minor members intersect or overlap at a joint, the order in which each member frames into the joint should be determined by wall thickness and/or diameter. The member with the thickest wall should be the continuous or through member, and the sequence for framing the remaining members shall be based on the order of decreasing wall thickness. If two or more members have the same wall thickness, the larger diameter member shall be the continuous or through member. If two or more members have the same diameter and wall thickness, either member may be the through member unless the designer has designated a through member. Sections of brace welds, which will be covered by other brace connections, shall be welded and the NDT (non-destructive testing) performed prior to cover up.

Where an increased brace wall thickness or higher yield or toughness properties is required for the brace, this material should extend beyond both the connection with the chord and the connection with any overlapping braces by the greater of a minimum of one brace diameter, or 600 mm.

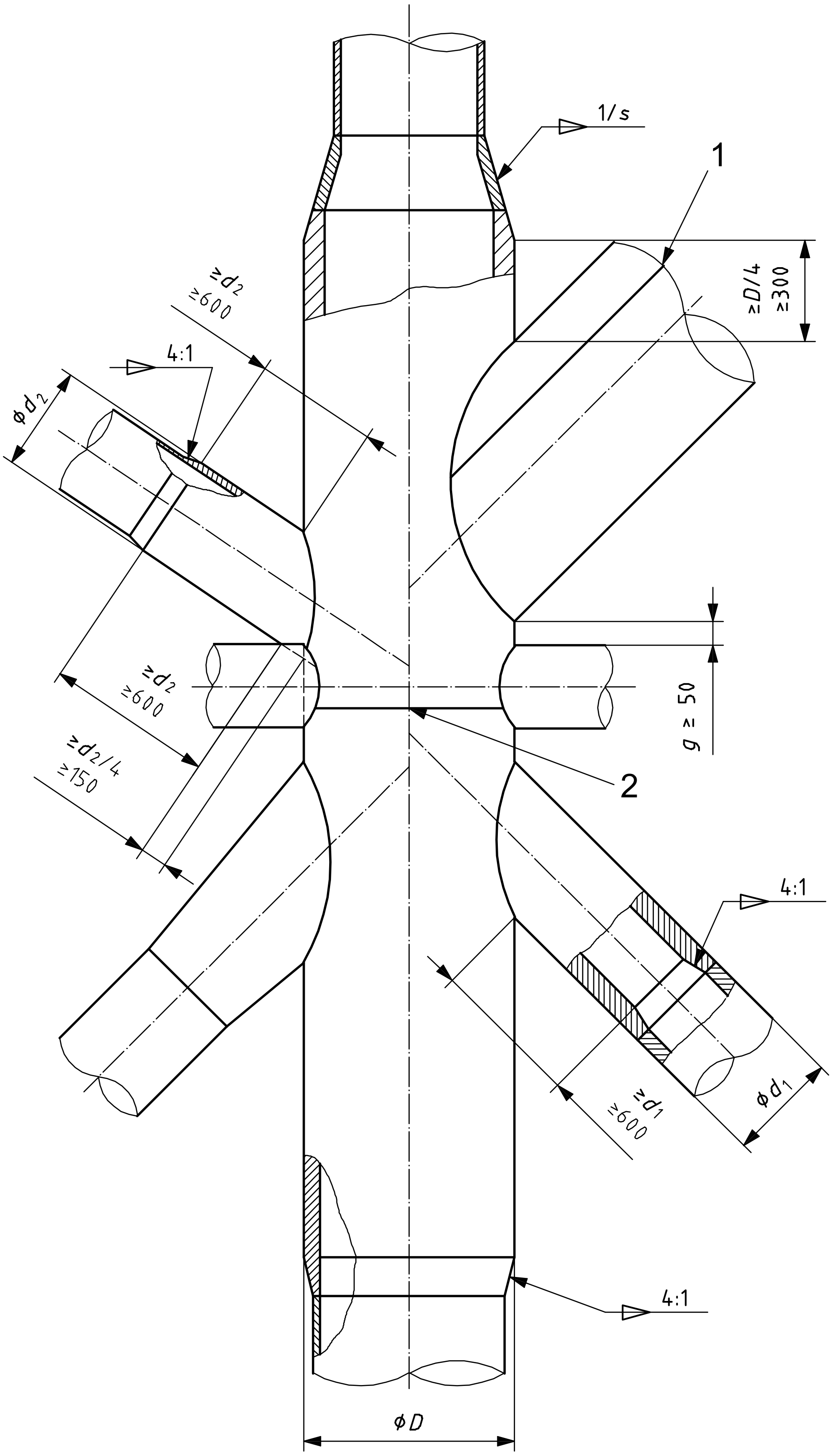
Neither the chord can nor the brace stub minimum dimensions given above include the length over which any thickness taper occurs; any difference in thickness between chord can and chord member or between brace stub and brace member shall be tapered at 1:4 or lower, see Figure 14.2-3. For joints where fatigue considerations are important, tapering on the inside can have both an undesirable influence on crack origin and make early detection of cracks more difficult; for such joints, tapering should be on the outside (i.e. matching internal diameters).

The nominal gap (i.e. excluding weld toes) between adjacent braces, whether in-plane or out-of-plane, should not be less than 50 mm. Overlapping of welds of non-overlapping braces at the weld toes shall be avoided. When braces overlap, the overlap should be at least $d/4$ (where d is the diameter of the through brace) or 150 mm, whichever is greater. This dimension is measured along the axis of the through member, see Figure 14.2-3.

Where braces overlap, the through brace shall have the thicker wall and shall be fully welded to the chord. Where there is a substantial overlap, the brace with the larger diameter should be the through member. The through brace can require an end stub to ensure that its thickness is at least equal to that of the overlapping brace.

Longitudinal seam welds and circumferential welds should be located to minimize or eliminate their impact on joint performance. The longitudinal seam weld of a brace should be located near the crown heel of the joint, see Figure 14.2-3. The longitudinal seam weld of the chord should be separated from incoming braces by at least 300 mm, see Figure 14.2-4. Where a chord requires a circumferential weld to achieve the desired can length, the weld should be positioned at a lightly loaded brace intersection, between saddle and crown locations, see Figure 14.2-3.

Dimensions in millimetres



Key

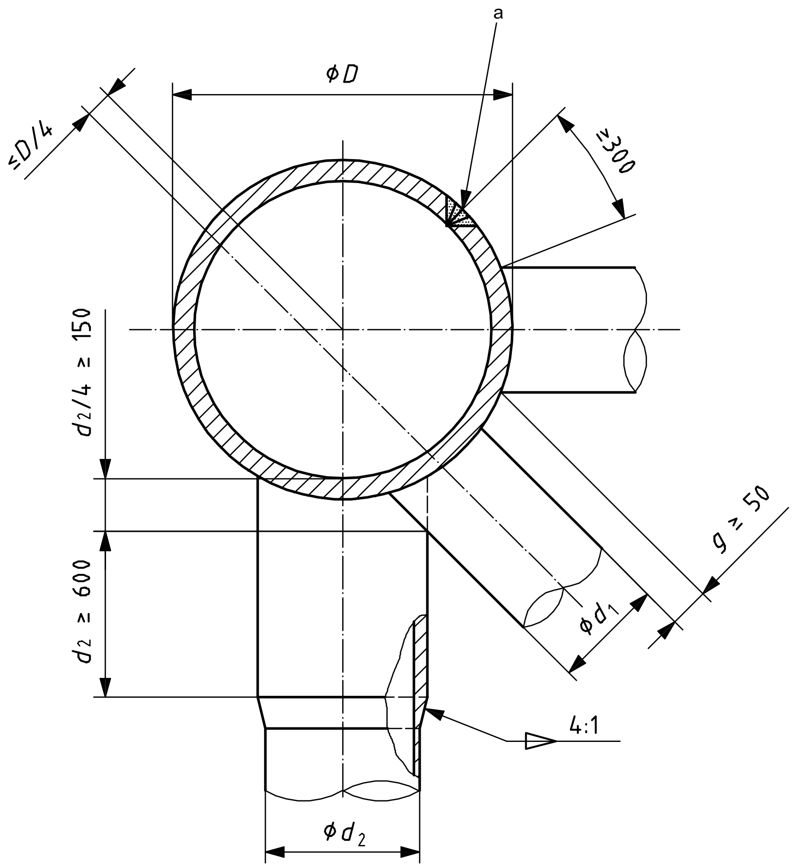
1 seam weld

2 can circumferential weld

s slope of conical transition ($s = 4$ for $D/T < 30$; flatter slope necessary for higher D/T ratios)

Figure 14.2-3 — In-plane joint detailing

Dimensions in millimetres



a Longitudinal seam weld of chord can.

Figure 14.2-4 — Out-of-plane joint detailing

14.3 Simple circular tubular joints

14.3.1 General

Simple tubular joints are joints having no gussets, diaphragms, grout or stiffeners. Simple Y- and X-joints have no overlap of principal braces, but simple K-joints may have overlaps up to 0,6 D.

The validity ranges for the formulae given in 14.3 are as follows:

$0,2 \leqslant \beta \leqslant 1,0$

$10 \leqslant \gamma \leqslant 50$

$30^\circ \leqslant \theta \leqslant 90^\circ$

$\tau \leqslant 1,0$

$f_y \leqslant 500$ N/mm^2

For K-joints, the following validity range also applies:

$g\ T > -1,2\gamma$

Annex A discusses approaches that may be adopted for joints which fall outside the above range.

14.3.2 Basic joint strength

The strength of a joint varies not only with its materials and geometry but also with the pattern of forces on each brace. Consequently, these strengths can vary between load cases.

The strengths for simple tubular joints subjected to axial brace forces or moments only should be calculated for each brace, for each individual force component of tension, compression, in-plane bending and out-of-plane bending, and for each load case consisting of a combination of forces.

Representative strengths for simple tubular joints are given in Equations (14.3-1) and (14.3-2):

$$P_{uj} = \frac{f_y \, T^2}{\sin\theta} Q_u \, Q_f \tag{14.3-1}$$

$$M_{uj} = \frac{f_y \, T^2 \, d}{\sin\theta} Q_u \, Q_f \tag{14.3-2}$$

where

P_{uj} is the representative joint axial strength, in force units;

M_{uj} is the representative joint bending moment strength, in moment units;

f_y is the representative yield strength of the chord member at the joint (SMYS or 0,8 of the tensile strength, if less), in stress units;

T is the chord wall thickness at the intersection with the brace;

d is the brace outside diameter;

θ is the included angle between brace and chord;

Q_u is a strength factor (see 14.3.3);

Q_f is a chord force factor (see 14.3.4).

For braces with a mixed classification, P_{uj} and M_{uj} should be calculated by weighting the contributions from Y-, K- and X-joint behaviour by the proportions of that behaviour in the joint. This means that P_{uj} and M_{uj} can be different for each load case considered, since joints can behave differently under different load cases, see A.14.3.2. However, for M_{uj} the values for Q_u for in-plane and out-of-plane moments are independent of the classification, see Table 14.3-1.

For joints with joint cans, P_{uj} shall not exceed the strength limits defined in 14.3.5.

The design strengths of simple tubular joints are

$$P_d = \frac{P_{uj}}{\gamma_{R,j}} \tag{14.3-3}$$

$$M_d = \frac{M_{uj}}{\gamma_{R,j}} \tag{14.3-4}$$

where

P_d is the design value of the joint axial strength, in force units;

M_d is the design value of the joint bending moment strength, in moment units;

$\gamma_{R,j}$ is the partial resistance factor for tubular joints, $\gamma_{R,j} = 1{,}05$.

14.3.3 Strength factor, Q_u

The strength factor, Q_u, varies with the joint classification and brace force type, as given in Table 14.3-1.

Table 14.3-1 — Values for Q_u

Joint classification	Brace force			
	Axial tension	Axial compression	In-plane bending	Out-of-plane bending
K	$(1{,}9+19\beta)Q_\beta^{0{,}5}Q_g$	$(1{,}9+19\beta)Q_\beta^{0{,}5}Q_g$	$4{,}5\ \beta\ \gamma^{0{,}5}$	$3{,}2\ \gamma^{(0{,}5\ \beta^2)}$
Y	$30\ \beta$	$(1{,}9+19\beta)Q_\beta^{0{,}5}$	$4{,}5\ \beta\ \gamma^{0{,}5}$	$3{,}2\ \gamma^{(0{,}5\ \beta^2)}$
X	$23\ \beta$ for $\beta \leqslant 0{,}9$ $20{,}7 + (\beta - 0{,}9)\ (17\ \gamma\ - 220)$ for $\beta > 0{,}9$	$[2{,}8+(12+0{,}1\gamma)\beta]Q_\beta$	$4{,}5\ \beta\ \gamma^{0{,}5}$	$3{,}2\ \gamma^{(0{,}5\ \beta^2)}$
Where Q_β and Q_g are given by Equations (14.3-5) to (14.3-8).				

The Q_u factor for tension forces for design is based on limiting the strength to first cracking. The Q_u factor that is associated with ultimate strength of Y- and X-joints for tension forces for use in assessment is given in A.14.3.3.

Q_β is a geometrical factor defined by

$$Q_\beta = \frac{0{,}3}{\beta(1-0{,}833\beta)} \qquad \text{for } \beta > 0{,}6 \qquad (14.3\text{-}5)$$

$$Q_\beta = 1{,}0 \qquad \text{for } \beta \leqslant 0{,}6 \qquad (14.3\text{-}6)$$

Q_g is a gap factor defined by

$$Q_g = 1{,}9 - 0{,}7\ \gamma^{-0{,}5}\ (g/T)^{0{,}5} \qquad \text{for } g/T \geqslant 2{,}0\text{, but } Q_g \geqslant 1{,}0 \qquad (14.3\text{-}7)$$

$$Q_g = 0{,}13 + 0{,}65\ \phi\ \gamma^{+0{,}5} \qquad \text{for } g/T \leqslant -2{,}0 \qquad (14.3\text{-}8)$$

for $-2{,}0 < g/T < +2{,}0$, the gap factor, Q_g, may be found by linear interpolation between the results of Equations (14.3-7) and (14.3-8) for the limiting values of $g/T = -2{,}0$ and $g/T = +2{,}0$.

where

$\phi = t \times f_{y,b}/(T \times f_y)$

and, in addition to the definitions given in 14.3.2,

$f_{y,b}$ is the representative yield strength of the brace at the intersection with the chord, in stress units;

t is the brace wall thickness at the intersection with the chord.

14.3.4 Chord force factor, Q_f

The chord force factor, Q_f, is a factor that accounts for the presence of forces from factored actions in the chord:

$$Q_f = 1{,}0 - \lambda\, q_A^2 \tag{14.3-9}$$

where λ is a factor dependent on force pattern and

λ = 0,030 for brace axial force;

= 0,045 for brace in-plane bending moment;

= 0,021 for brace out-of-plane bending moment.

The parameter, q_A, is defined as follows:

$$q_A = \left[C_1 \left(\frac{P_C}{P_y} \right)^2 + C_2 \left(\frac{M_C}{M_p} \right)^2_{ipb} + C_2 \left(\frac{M_C}{M_p} \right)^2_{opb} \right]^{0,5} \gamma_{R,q} \tag{14.3-10}$$

where

P_C is the axial force in the chord member from factored actions;

M_C is the bending moment in the chord member from factored actions;

P_y is the representative axial strength due to yielding of the chord member not taking account of buckling, in force units

$$P_y = A\, f_y$$

f_y is the representative yield strength of the chord member, in stress units;

A is the cross-sectional area of the chord or chord can at the brace intersection;

M_p is the representative plastic moment strength of the chord member;

$\gamma_{R,q}$ is the partial resistance factor for yield strength, $\gamma_{R,q}$ = 1,05;

ipb refers to in-plane bending;

opb refers to out-of-plane bending;

C_1, C_2 are the coefficients given in Table 14.3-2.

Table 14.3-2 — Values for the coefficients C_1 and C_2

Joint type	C_1	C_2
Y-joints for calculating strength against brace axial forces	25	11
X-joints for calculating strength against brace axial forces	20	22
K-joints for calculating strength against balanced brace axial forces	14	43
All joints for calculating strength against brace moments	25	43

When calculating the chord force factor, Q_f, the higher value of q_A for the chord on either side of the brace intersection shall be used.

For K-joints, chord axial tension forces may be ignored when calculating Q_f.

14.3.5 Y- and X-joints with chord cans

For simple Y- and X-joints with a chord can, the joint representative axial strength shall be calculated using Equation (14.3-11):

$$P_{uj} = \left[r + (1 - r)(T_n / T_c)^2 \right] P_{uj,c} \tag{14.3-11}$$

where

P_{uj} is the representative joint axial strength, in force units;

$P_{uj,c}$ is the value of P_{uj} from Equation (14.3-1), based on chord can geometrical and material properties, including Q_f calculated from chord can properties and dimensions;

r $= L_c/(2{,}5\ D)$ for joints with $\beta \leqslant 0{,}9$;

$= (4\ \beta - 3)\ L_c/(1{,}5\ D)$ for joints with $\beta > 0{,}9$;

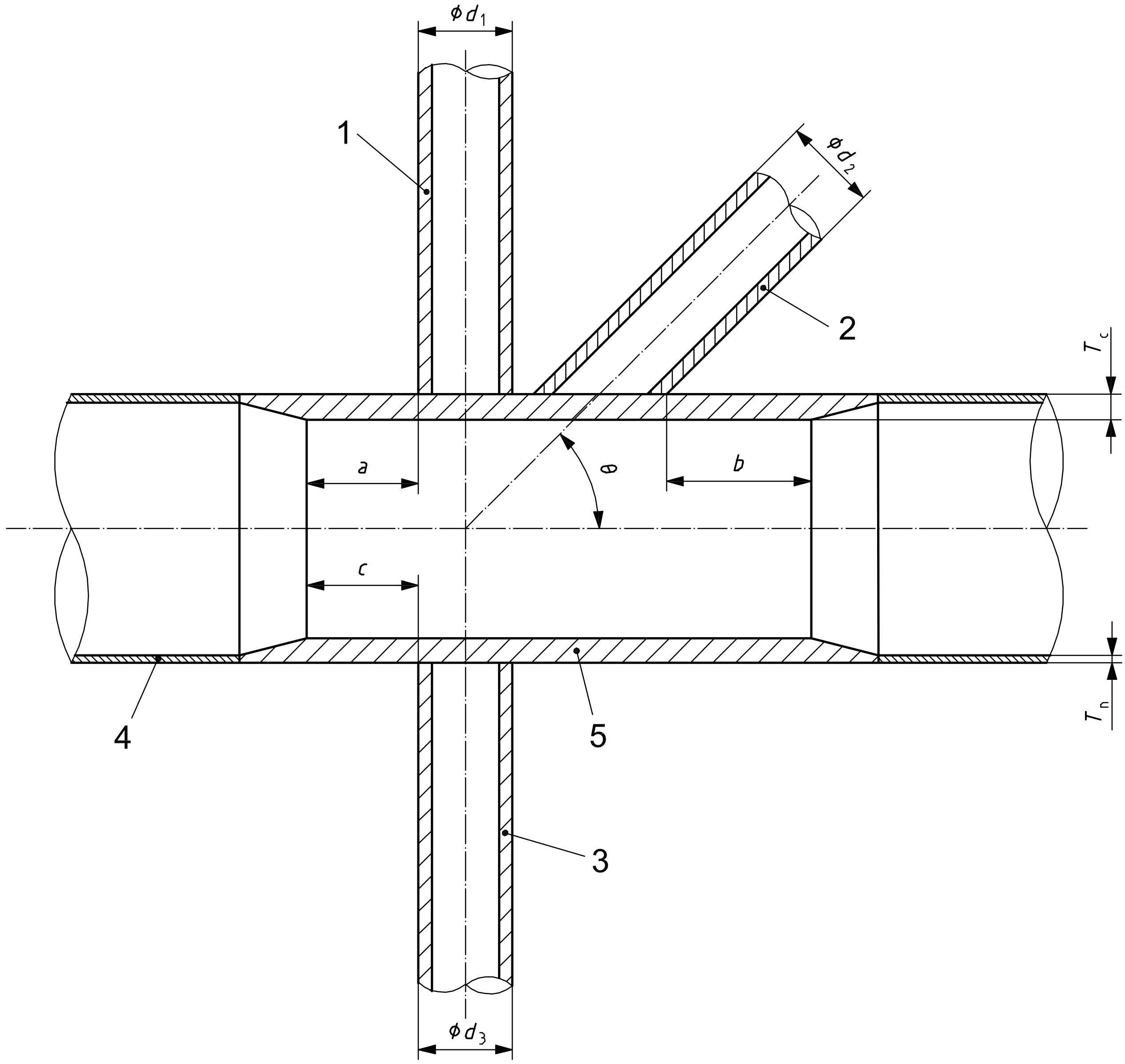
L_c is the effective total length, see Figure 14.3-1;

T_n is the lesser of the chord member thicknesses on either side of the joint, see Figure 14.3-1;

T_c is the chord can thickness, see Figure 14.3-1.

In no case shall r be taken as greater than unity. Figure 14.3-1 gives examples for the calculation of L_c.

Alternatively, an approximate closed ring analysis may be undertaken. Such an analysis should include plastic analysis with appropriate safety factors, and an effective chord length up to 1,25 D on either side of the line of action of the branch forces at the chord face, taking into account any thickness changes within this distance. Where multiple branches are in the same plane, and dominantly loaded in the same sense (tension or compression), the relevant perpendicular force is summed over all the braces on one side.



Key

1 brace 1
2 brace 2
3 brace 3
4 nominal chord
5 chord can

Calculation of effective total length

Brace	L_c
1	$2a + d_1$
2	$2b + d_2/\sin\theta$
3	$2c + d_3$

Figure 14.3-1 — Examples of chord length L_c calculation

14.3.6 Strength check

Each brace in a joint that is subjected either to an axial force or a bending moment alone, or to an axial force combined with bending moments, shall be designed to satisfy the following conditions:

$$U_j = \left|\frac{P_B}{P_d}\right| + \left(\frac{M_B}{M_d}\right)^2_{ipb} + \left|\frac{M_B}{M_d}\right|_{opb} \leqslant 1{,}0 \qquad \text{for all joints} \qquad (14.3\text{-}12)$$

$$U_\mathrm{j} = \left|\frac{P_\mathrm{B}}{P_\mathrm{d}}\right| + \left(\frac{M_\mathrm{B}}{M_\mathrm{d}}\right)^2_\mathrm{ipb} + \left|\frac{M_\mathrm{B}}{M_\mathrm{d}}\right|_\mathrm{opb} \leqslant \frac{U_\mathrm{b}}{\gamma_\mathrm{zj}} \quad \text{for all joints except those identified as non-critical} \qquad (14.3\text{-}13)$$

where

U_j is the joint utilization;

P_B is the axial force in the brace member from factored actions;

M_B is the bending moment in the brace member from factored actions;

P_d is the design value of the joint axial strength (see 14.3.2);

M_d is the design value of the joint bending moment strength (see 14.3.2);

ipb represents in-plane bending moments and strengths;

opb represents out-of-plane bending moments and strengths;

U_b is the calculated brace utilization from the applicable brace interaction equation checks from Clause 13, the reduced limit on forces applies to critical joints only, see 14.2.3;

γ_zj is the extra partial resistance factor from Equation (14.2-2).

14.4 Overlapping circular tubular joints

Overlapping joints are joints where braces overlap in-plane or out-of-plane at the chord member surface. Figures 14.2-3 and 14.2-4 include both non-overlapping and overlapping braces.

The strength of joints that have in-plane overlap involving two or more braces may be determined using the requirements for simple joints defined in 14.3, with the following exceptions and additions.

a) Shearing of the brace parallel to the chord face is a potential failure mode and shall be checked.

b) Subclause 14.3.5 does not apply to overlapping joints.

c) If axial forces in the overlapping and through braces have the same sign (both in compression or both in tension), the check of the intersection strength of the through brace on the chord shall use the combined axial force representing the force in the through brace plus the portion of the overlapping brace force(s). The portion of the overlapping brace force may be calculated from the ratio of the cross-sectional area of the brace that bears onto the through brace to the full area of the overlapping brace.

d) For both in-plane or out-of-plane moments, the combined moments on the overlapping and through braces shall be used to check the intersection strength of the through brace on the chord. This combined moment shall account for the sign of the moments.

e) The overlap onto the through brace shall be checked by using the through brace as the chord in the equations in 14.3. The through brace strength shall also be checked for combined axial force and bending moment in the overlapping brace in accordance with 14.3.6 using the value of Q_f calculated for the through brace.

f) Where nominal thicknesses of the overlapping and through braces differ by more than 10 %, the thicker brace shall be the through brace.

Joints having out-of-plane overlap may be assessed on the same general basis as in-plane overlapping joints, except that the axial strength should normally revert to that for Y-joints.

14.5 Grouted circular tubular joints

Two varieties of grouted joints are commonly used in practice. The first is a fully grouted chord, the second has a double-skin, whereby grout is placed in the annulus between an outer chord and an internal member. In both cases, the grout is unreinforced and no benefit to joint strength shall be derived from shear keys (if present).

The strength of grouted joints that are otherwise of simple configuration may be determined using the simple joint requirements defined in 14.3, with the modifications and limitations below.

a) For double-skin grouted joints, an effective thickness, T_e, should be determined from Equation (14.5-1):

$$T_e = \left(T^2 + T_p^2\right)^{0,5} \tag{14.5-1}$$

where

T is the wall thickness of the chord;

T_p is the wall thickness of the inner member.

The effective thickness, T_e, should be used in place of T in the simple joint equations, including the determination of the γ parameter, but taking into account b) and c) hereafter.

m) The Q_u values given in Table 14.3-1 should be replaced with values pertinent to grouted joints determined from Table A.14.5-1. As the expressions in Table A.14.5-1 are based on tear-out of the chord wall, the chord wall thickness, T, should be used instead of the effective thickness, T_e. Q_u values should not be less than those for simple ungrouted joints with the same geometry and forces. Joint can derating for chord can length of Y- and X-joints (see 14.3.5) may be disregarded for fully grouted joints, but should be included for double-skin grouted joints with thin skins.

n) The Q_f value calculated from Equations (14.3-9) and (14.3-10) depends on joint type and brace loading through the coefficients in Table 14.3-2, but not on thickness.

o) Axial and moment design strengths calculated for the joint shall exceed the brace design strengths, for non-compact members inclusive of the local buckling strength at the brace end.

14.6 Ring stiffened circular tubular joints

Primary joints along launch frames are often strengthened by ring stiffening. Ring stiffening is also used in some structures to address fatigue requirements or to avoid very thick chord cans. A.14.6 outlines the salient features of several common approaches available to design ring-stiffened joints.

14.7 Other circular joint types

Joints not covered by 14.3 to 14.6 may be designed on the basis of appropriate experimental, numerical, or in-service evidence. Material strength approaches may be employed, although extreme care is needed in identifying all components that are expected to participate in resisting incoming brace forces and in establishing the acting force envelopes prior to conducting strength checks. Often material strength checks are complemented with calibrated FEA (finite element analysis) to establish the magnitude and location of acting stresses.

14.8 Damaged joints

Joints in existing structures sometimes become damaged as a result of fatigue, corrosion, or overload (environmental or accidental). In such cases, the reduced joint strength shall be estimated either from simple models, e.g. based on the use of reduced area or section modulus, or else shall be based on more extensive numerical analysis using FEA models or experimental evidence.

14.9 Noncircular joints

Connections with noncircular chord or brace sections are typically used in topsides structures, see ISO 19901-3[2]. Common types include I beam, column, and plate girder sections and rectangular or square hollow sections. For some arrangements, detailed land-based design practice is available, see A.14.9. For arrangements for which little or no practice is available, the requirements are similar to those according to 14.7.

14.10 Cast joints

Cast joints can be of any geometry and are usually of variable wall thickness; they are of particular benefit when there are many similar joints or where fatigue is causing difficulties. The design of a cast joint requires calibrated FEA. A conservative design approach for strength is to limit stresses in the joint due to forces from factored actions to below the design yield strength of the material, $f_y/\gamma_{R,q}$ (see 14.3.4). The design of cast joints is often carried out in conjunction with the manufacturer.

15 Strength and fatigue resistance of other structural components

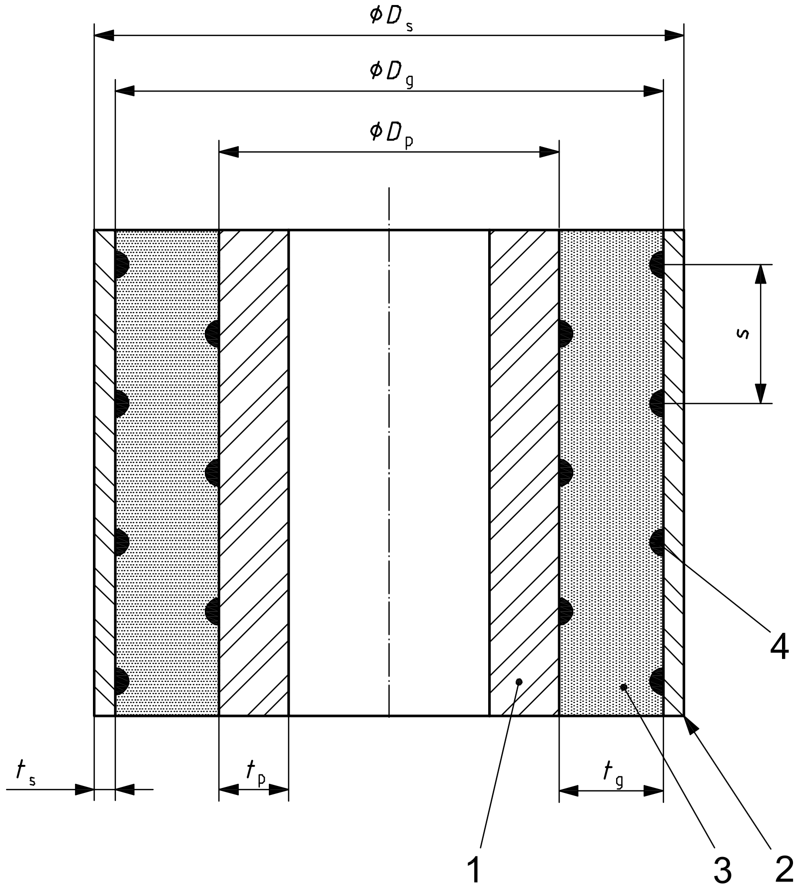
15.1 Grouted connections

15.1.1 General

Forces can be transferred between concentric tubular members by grouting the annulus between them. These forces are transferred from steel member to steel member, across the steel to grout interfaces and through the grout. Such connections are most commonly used for foundation pile-to-sleeve connections, but are also used for other types of connections, such as connecting pins and repairs.

The grout shall comply with the requirements of 19.6. Fabrication and installation shall be in accordance with 20.4.5 and 22.5.12, respectively.

Within 15.1, the nomenclature and symbols shown in Figure 15.1-1 and Figure 15.1-2 are used.



Key

1 inner steel tubular member (pile in foundation pile-to-sleeve connection)

2 outer steel tubular member (sleeve in foundation pile-to-sleeve connection)

3 grout

4 shear keys (if present)

D_s outside diameter of the outer steel tubular member (the sleeve)

D_g outside diameter of grout annulus

D_p outside diameter of the inner steel tubular member (the pile)

t_s wall thickness of the outer steel tubular member (the sleeve)

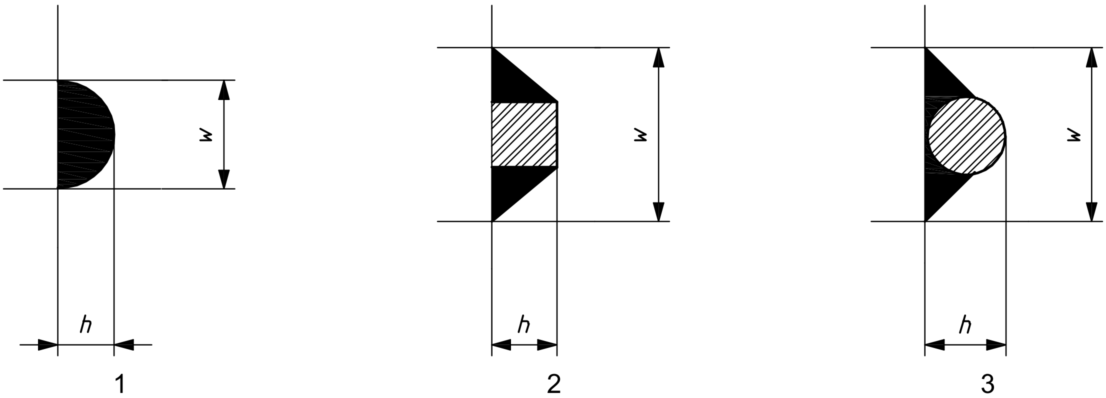
t_g nominal thickness of the grout annulus (i.e. assuming inner and outer members are concentric)

t_p wall thickness of the inner steel tubular member (the pile)

s centre-to-centre spacing of the shear keys

Figure 15.1-1 — Grouted connection symbols and terminology

Where shear keys are used, they shall be of one of the forms shown in Figure 15.1-2.



Key

1 weld bead

2 square bar with fillet beads

3 round bar with fillet beads

h shear key height

w shear key width

Figure 15.1-2 — Shear key detail options

15.1.2 Detailing requirements

Centralisers shall be used to maintain a sufficient annular space between the two tubulars. For pile-to-sleeve connections, the nominal width of the annulus shall be 40 mm or greater when a cement and water grout is used. In other types of connections, and when fillers are used in the grout, the width of the annulus shall be sized appropriately to enable free and uniform grout flow.

Proper means for the introduction of the grout slurry into the annulus shall be provided so that the potential for the dilution of the grout and formation of voids is minimized. The grouting arrangements shall be such that grout enters at the bottom of the annulus, and displaces water upwards as it fills the annulus.

Packers or other forms of end seals shall, as a minimum, be used at the lowest end of a connection. Provisions should, if necessary, be made for setting a grout plug in the event that an end seal fails. At sites having soft sea beds the use of wipers or other means of minimizing mud intrusion should be provided for pile-to-sleeve connections.

15.1.3 Axial force

The axial force, P, to be used for the design of a grouted connection, shall be the largest axial force on the connection (either tensile or compressive) determined from factored actions.

In determining P for pile-to-sleeve (foundation) connections, consideration shall be given to soil-pile-structure interaction. In addition, account shall be taken of the distribution of overall actions on the structure over various piles in a group or cluster.

15.1.4 Interface transfer stress

When a grouted connection is subjected to combined axial force and a torsional moment, the interface transfer stress, σ_p, shall be taken as the resultant of the component stresses caused by axial force and torsional moment at the inner member.

The component of the interface transfer stress at the inner steel tubular member due to axial force, σ_a, is defined by Equation (15.1-1):

$$\sigma_a = \frac{P}{\pi D_p L_e} \tag{15.1-1}$$

where

P is the axial force on the grouted connection due to factored actions;

D_p is the outside diameter of the inner steel tubular member (the pile);

L_e is the effective grouted connection length.

The effective grouted connection length shall be calculated as the overall length of the grout along the axis of the members, reduced by the following non-structural lengths.

a) Where setting a grout plug is the primary means of sealing or is the contingency sealing method in the event of packer failure, the grout plug length shall be considered as non-structural.

b) To allow for potentially weak interface zones, grout slump, etc., at each end of the connection, the greater of the following grouted lengths shall be considered as non-structural:

   1) two thicknesses of the grout annulus, $2t_g$;

   2) one shear key spacing, s, if shear keys are used.

p) Any grouted length that is not certain to contribute effectively to the connection strength shall be considered as non-structural.

   EXAMPLE When shear keys are used, the implications of possible over- and under-driving of piles (foundation) are required to be considered in relation to the number of keys present in the grouted length.

The component of the interface transfer stress at the inner steel tubular member due to torsion, σ_t, is defined as follows:

$$\sigma_t = \frac{M_t}{\pi D_p^2 L_e} \quad (15.1\text{-}2)$$

where

M_t is the torsional moment on the grouted connection due to factored actions;

D_p is the outside diameter of the inner steel tubular member (the pile).

The combined interface transfer stress (σ_p) is the stress resultant of both components:

$$\sigma_p = \sqrt{\sigma_a^2 + \sigma_t^2} \quad (15.1\text{-}3)$$

where σ_p is the interface transfer stress at the inner steel tubular member (the pile).

15.1.5 Interface transfer strength

15.1.5.1 General

The design interface transfer strength, f_d, for cement-water grouts shall be calculated from the lesser of the representative interface transfer strengths for sliding at the grout-steel interface, $f_{g,sliding}$, and the representative interface transfer strength for grout matrix failure, $f_{g,shear}$:

$$f_d = \frac{f_g \, k_{red}}{\gamma_{R,g}} \quad (15.1\text{-}4)$$

where

f_g is the representative interface transfer strength and is the lesser of $f_{g,sliding}$ and $f_{g,shear}$;

k_{red} is a reduction factor for the effects of movement during grout setting, see 15.1.5.3;

$\gamma_{R,g}$ is the partial resistance factor for interface transfer strength

$\gamma_{R,g} = 2{,}0$

If the representative interface transfer strength is based on test results, rather than the formulae below, account shall be taken of the inherent variability in the data when calculating the representative interface strength, see 7.7.

The representative interface transfer strength for grout-steel interface sliding ($f_{g,sliding}$) for the range of validity of the parameters given in 15.1.5.2 is calculated using Equation (15.1-5):

$$f_{g,sliding} = C_p \left[2 + 140 \left(\frac{h}{s} \right)^{0,8} \right] K^{0,6} f_{cu}^{\ 0,3} \qquad (15.1\text{-}5)$$

The representative interface transfer strength for grout matrix shear failure ($f_{g,shear}$) for the range of validity of the parameters given in 15.1.5.2 is calculated using Equation (15.1-6):

$$f_{g,shear} = \left[0{,}75 - 1{,}4 \left(\frac{h}{s} \right) \right] f_{cu}^{\ 0,5} \qquad (15.1\text{-}6)$$

where

C_p is a scale factor for the diameter of the inner steel member (the pile)

for $D_p \leqslant 1\,000$ mm $\qquad C_p = \left(D_p / 1\,000 \text{ mm} \right)^2 - \left(D_p / 500 \text{ mm} \right) + 2 \qquad$ (15.1-7)

for $D_p > 1\,000$ mm $\qquad C_p = 1{,}0 \qquad$ (15.1-8)

s is the shear key spacing;

h is the shear key height;

f_{cu} is the specified unconfined cube strength of the grout as defined in 19.6.2;

K is the radial stiffness factor of the grouted connection

$$K = \left[\left(\frac{D_p}{t_p} \right) + \left(\frac{D_s}{t_s} \right) \right]^{-1} + \frac{1}{m} \left(\frac{D_g}{t_g} \right)^{-1} \qquad (15.1\text{-}9)$$

where m is the ratio of elastic moduli of steel and grout $= E_s/E_g$ (to be taken as 18 in lieu of actual data).

Equations (15.1-5) and (15.1-6) may also be used for grouts containing sand aggregate in lieu of more specific information; however, the strengths calculated will be conservative and the use of test results and of other data for sand-cement-water grouts is recommended.

15.1.5.2 Ranges of validity

Equations (15.1-5) and (15.1-6) are valid for non-machined and uncoated tubular members, where mill scale has been fully removed by corrosion or mechanical means, and for the following ranges of associated parameters.

— The specified minimum cube strength shall be within the following range:

$$20 \text{ MPa} \leqslant f_{cu} \leqslant 80 \text{ MPa}$$

A specified minimum 28 day grout cube strength of 40 MPa is recommended for the design of new structures.

— The nominal geometric dimensions of the grouted connection shall be within the following ranges:

$$1{,}5 \leqslant w/h \leqslant 3{,}0$$

$$0{,}0 \leqslant h/s \leqslant 0{,}10$$

$$20 \leqslant D_p/t_p \leqslant 40$$

$$30 \leqslant D_s/t_s \leqslant 140$$

$$10 \leqslant D_g/t_g \leqslant 45$$

$$h/D_p \leqslant 0{,}012$$

$$D_p/s \leqslant 16$$

$$1 \leqslant L_e/D_p \leqslant 10$$

— The scale factor for the diameter of the inner steel member and the radial stiffness factor of the grouted connection shall be

$$C_p \leqslant 1{,}5$$

$$K \leqslant 0{,}02$$

— Shear keys, where used, shall be continuous hoops or a continuous helix. Hoop shear keys shall be uniformly spaced, orientated perpendicular to the axes of the tubular members and be of the same form, height and spacing on both the inner and the outer tubular members. Where helical shear keys are used, the representative interface transfer strengths, given by Equations (15.1-5) and (15.1-6), shall be reduced by a factor of 0,75 and the following additional limitation shall be applied:

$$\frac{D_p}{s} \geqslant 2{,}5$$

15.1.5.3 Effect of movements during grout setting

The possible movements between the inner and outer steel tubular members during the 24 h period after grouting shall be determined for the maximum expected sea state during that time, assuming the grout makes no contribution to the stiffness of the system. For foundation pile-to-sleeve connections, this analysis shall be an on-bottom analysis of the structure with ungrouted piles.

The reduction factor, k_{red}, for the effects of movement during grout setting shall be calculated as follows.

— For a relative axial movement between the steel tubular members of less than 0,035 % of D_p:

$$k_{red} = 1{,}0 \tag{15.1-10}$$

— For a relative axial movement between the steel tubular members of between 0,035 % of D_p and 0,35 % of D_p and for $h/s \leqslant 0{,}06$

$$k_{red} = 1{,}0 - 0{,}1\,(h/s)\,f_{cu} \tag{15.1-11}$$

— For a relative axial movement between the steel tubular members of between 0,035 % of D_p and 0,35 % of D_p and for $h/s \geqslant 0{,}06$, see A.15.1.5.3.

This reduction factor is valid for relative movements between steel tubular members that do not exceed 0,35 % of D_p. Movements in excess of 0,35 % of D_p cause higher levels of strength degradation and shall be avoided.

15.1.6 Strength check

The combined interface transfer stress (σ_p) from Equation (15.1-3) shall satisfy the following strength check:

$$U_{g,c} = \frac{\sigma_p}{f_d} \leqslant 1{,}0 \qquad (15.1\text{-}12)$$

where

$U_{g,c}$ is the utilization of the grouted connection for combined axial force and torsion;

f_d is the design interface transfer strength from Equation (15.1-4).

The component of the interface transfer stress at the inner member due to torsion (σ_t) from Equation (15.1-2) shall additionally satisfy the following check:

$$U_{g,t} = \frac{\sigma_t}{f_{d,0}} \leqslant 1{,}0 \qquad (15.1\text{-}13)$$

where

$U_{g,t}$ is the utilization of the grouted connection for torsion alone;

$f_{d,0}$ is the design interface transfer strength from Equation (15.1-4), calculated for a plain grouted connection with no shear keys, i.e. with $h/s = 0$.

In bending and shear, the grout principally acts as a confined spacer transferring compressive stresses only. The representative interface strength for axial force is thus not reduced by coexisting bending and shear. However, consideration shall be given to the possibility of local buckling of the free end of the internal tubular member.

The steel components of the connection shall be designed in accordance with Clauses 13, 14 and 17, with 15.2 and 15.3, and by testing and analysis in accordance with Clause 7, as appropriate.

15.1.7 Fatigue assessment

Grouted connections that comply with the static strength requirements of 15.1.5 and 15.1.6, and that are subjected primarily to cyclically varying forces due to wave action do not require a detailed fatigue assessment.

In other cases, the fatigue strength may be assumed to be adequate if in any force cycle:

— the direction of the maximum axial stress is determined (e.g. tension or compression in the tubular steel members beyond the end of the grout);

— the maximum axial stress in the opposite direction ($\sigma_{a,rev}$) is determined (i.e. compression or tension respectively);

— the combined interface transfer stress (σ_p) calculated using $\sigma_{a,rev}$ in Equation (15.1-3) satisfies Equation (15.1-12) with $f_d = f_{d,0}$, i.e. for f_d calculated for a plain grouted connection with no shear keys ($h/s = 0$).

If necessary, the effective grouted connection length should be increased to achieve these requirements or a detailed evaluation should be carried out. A.15.1.7 provides guidance on this.

15.2 Mechanical connections

15.2.1 Types of mechanical connectors

Mechanical connectors may be used to join structural components of offshore structures. These components include, but are not limited to, (foundation) piles, conductors, production and export risers, and tendons. Although there are a variety of connector designs, the following four general types of connectors are most commonly used for joining coaxial tubular members:

a) threaded or grooved connectors of the pin-and-box type assembled by torque or radial interference;

b) flanged connectors assembled by threaded fasteners;

c) dogged connectors that use radial wedges between the pin and box parts;

d) swaged connectors.

Requirements for conventional mechanical connectors a) to c) above are given in 15.2.2 to 15.2.7; specific requirements for threaded connections only are given in 15.2.8; and requirements for swaged connections are given in 15.2.9.

Connectors that rely on impinging grip or friction grip by means of collets or sliding wedges to effect load transfer are generally not amenable to the analysis procedures discussed in A.15.2. However, the structural and validation requirements still apply and testing is recommended for validating performance.

15.2.2 Design requirements

15.2.2.1 General

The design of mechanical connectors for structural applications shall comply with the following structural and functional requirements. Structural requirements relate to the static strength and fatigue performance, while functional requirements include assembly, disassembly, installation, and sealing.

15.2.2.2 Static strength requirements

Connectors should be as strong as, or stronger than, the weakest of the connected tubular members. In all cases, connectors shall be designed to the strength criteria in 15.2.5. If the connector has a strength lower than the representative yield strength of the attached full tubular cross-section, the effect of relevant extreme forces imparted by waves and current, as well as forces due to deformation of the connector, forces due to possible restraints, and accidental actions shall all be considered. The effects of deformations and sudden disengagement, such as thread jump-out, shall also be considered in detail.

15.2.2.3 Fatigue performance requirements

Connectors should have a fatigue endurance in excess of full penetration, circumferential welds between tubular members of the same size. Failing this, the fatigue life of the connector shall exceed the planned life for the tubular component by a fatigue damage design factor, see 16.12.2, taking into consideration

— inspectability and inspection programme (see Clause 23),

— likelihood and consequence of failure,

— uncertainty in forces due to actions and analysis methods,

— uncertainty in the fatigue life calculation method, and

— remaining life, if through-thickness cracking does occur.

For structurally critical applications, where in-service inspection of the connector is unreliable or not possible, the calculated fatigue life of the connector shall be a minimum of 10 times the planned life of the connected tubular members.

15.2.2.4 Functional requirements

Connectors shall assemble and disassemble (when applicable) in a reliable manner consistent with the assumptions made in the analysis of the model used to demonstrate compliance with both strength and fatigue criteria. In cases where the connector is to contain internal or external pressure, seals shall be provided such that they remain sound after installation and for the design life of the connection. All necessary functions of the connector shall remain effective upon application of the design and installation forces.

15.2.3 Actions and forces on the connector

Actions can be applied directly to a connector or indirectly through the tubular members being connected. The actions can include any combination of pressures (internal, external or both), forces and moments in the connected tubular members, deformations and thermal gradients. A simplified approach for calculating an equivalent axial force on the tubular members when they are simultaneously subjected to axial forces and bending moments is given in A.15.2.3.

15.2.4 Resistance of the connector

The strength of the connector can be determined by a calibrated analysis (see A.15.2.4) or prototype testing to failure (see 7.7). The strength shall be determined as the maximum applied typical force that can be sustained by the fully made up connector in combination with all other relevant deformations and actions applied repeatedly and in any order, without giving rise to gross deformation, disengagement, or, if applicable, loss of sealing. The representative resistance of the connector, R_c, shall be derived in accordance with 7.7.

15.2.5 Strength criteria

Connectors shall be designed to sustain the forces due to factored actions and deformations arising from external actions applied to the tubular members, thermal gradients, internal and external pressures, and assembly, without exceeding the representative resistance of the connector as defined in 15.2.4 and reduced by the partial resistance factor γ_R. The potential effect of temperature on material strength shall be taken into account. One approach to conducting strength checks for connectors based on linear FEA is given in A.15.2.5.

15.2.6 Fatigue criteria

To satisfy the requirements given in 15.2.2.3, the fatigue life of the connector needs to be determined. For an approximate fatigue life calculation, either of two methods may be used: the stress life approach (the S–N curve method) or the strain life approach (the initiation life method). Either approach requires local elastic stresses at the fatigue critical location(s) in the connector induced by the forces in the connected tubular members and material-specific fatigue data (see A.15.2.6).

15.2.7 Stress analysis validation

15.2.7.1 General

Although FEA programs are used to analyse connectors, some degree of experimental verification of the results obtained is highly desirable for confirming the validity of boundary conditions, simplifications in geometry, and preload simulation. Hence, direct measurements of actual strains/stresses using experimental techniques, e.g. strain gauges or photo elasticity, shall be included in a validation programme.

15.2.7.2 Strength validation

Experimental validation of the static strength of connectors shall either

a) demonstrate that the connector is stronger than the weakest of the connected tubular members, or

b) establish the actual strength of the connector.

The advantage of b) is that it gives a measure of the actual reserve strength of the connector. In planning strength tests, specimen size and force-moment-pressure (P-M-p) interaction shall be considered in light of the requirements imposed by the particular application.

15.2.7.3 Fatigue validation

Because of the many uncertainties involved in fatigue calculation methods and the nature of the phenomenon itself, prototype tests shall be conducted to confirm the fatigue performance of the connector, unless equivalent life-time service records are available for the connector in question. Uncertainties arise from effects that are difficult, if not impossible, to account for in the analysis procedures, but which can have a significant impact on fatigue life. Examples are connector size, surface finish, metallurgical and environmental effects. The extent and conditions of the fatigue testing should be commensurate with the level of conservatism built into the design process, the refinement of the analytical tools, the criticality of the connector application, and the degree of reliability sought through the validation exercise.

15.2.7.4 Functionality validation

Analytical modelling of variables, such as surface contact, surface roughness and friction, is difficult and unreliable. These variables affect the functional performance of connectors in terms of assembly and sealing. Thus, tests shall be conducted to demonstrate that connectors meet the functionality requirements. The tests shall use full-scale prototypes under conditions equivalent to, or more stringent than, those expected during operation.

15.2.8 Threaded fasteners

15.2.8.1 General

In some structural components, such as flanges and clamps, threaded fasteners are the primary means to transfer forces across the assembled mechanical connector. Threaded fasteners should normally be pre-tensioned, in order to provide resistance to disengagement and improve fatigue performance. Stresses in the threaded fastener induced by preload alone should be treated as secondary, see A.15.2.5.1.

15.2.8.2 Threaded fastener materials and manufacturing

Threaded fastener materials shall be selected taking account of the application of the connector. Factors to be considered include

— maximum stress level to which the fastener is subjected,

— material yield strength, ultimate strength, and ductility,

— corrosion resistance and galvanic corrosion,

— durability at service temperature in the offshore environment,

— selection of suitable coatings to provide the resistance to corrosion,

— embrittlement, and

— loss of stress in the fastener due to relaxation and creep.

The yield strength of carbon steel threaded fasteners should be less than 725 MPa, to avoid potential embrittlement due to hydrogen absorption from exposure to cathodic protection potentials.

Manufacture of threaded fastenings shall take account of the intended service, see A.15.2.8.2.

15.2.8.3 Threaded fastener installation

To ensure that the threaded fasteners achieve their design tension, a suitable installation technique is required. A comprehensive installation methodology shall be derived and documented. Simultaneous use of hydraulic jacks or tensioners, one for each threaded fastener, is recommended. This technique avoids uneven preload, which can result from sequential stressing techniques. For clamps where torquing devices are used, the accuracy of the relation between applied torque and tension shall be allowed for during installation (see A.15.2.8.3).

When using tensioners, the installation stress shall allow for transfer losses due to immediate elastic strains in the fastener thread and structural components that occur when the preload transfers from the tensioner to the threaded fastener. The installation stress shall also allow for longer term losses caused by creep in the threaded fastener and clamp components and for shrinkage of grout annuli or liner where used.

Spherical washers can relieve bending of the threaded fastener due to initial nut/bearing and plate misalignment.

Methods to prevent loosening of the fasteners shall be considered.

15.2.8.4 Threaded fastener inspection

Threaded fasteners should be periodically inspected during operation to ensure that they remain sound, protected from corrosion and properly tensioned. The inspection of the threaded fasteners should include measurements with respect to cathodic protection and tension (see A.15.2.8.4 and Clause 23).

The inspection of threaded fasteners may take into account any redundancy provided by the number of threaded fasteners.

15.2.8.5 Threaded fastener strength criteria

Threaded fasteners shall be designed to sustain the combined effect of assembly preload and the factored actions applied to the connected tubular members.

15.2.8.6 Threaded fastener fatigue criteria

The fatigue performance of threaded fasteners is directly linked to the stress ranges induced in them by the variation of the applied actions. Such stress ranges shall be calculated by accepted engineering methods, taking into account the effect of preload, and the relative stiffness of the fasteners and the structural component parts they fasten. Palmgren-Miner's rule, in conjunction with stress histories and S–N curves, based on tests of actual fasteners, may be used to calculate fatigue damage.

15.2.9 Swaged connections

15.2.9.1 Strength of swaged connections

A swaged connection between concentric tubular members is made by radially expanding, or swaging, the inner tubular into circumferential grooves machined on the internal surface of the outer tubular. Upon swaging, a permanent radial preload and a longitudinal mechanical interlock develop between the inner and the outer tubular walls. The resulting connection is referred to as a swaged connection.

Swaged connections shall be designed to sustain the static forces in the tubular members with no significant relative axial slippage between the inner member and the outer member, and no permanent deformation upon loading the connection within its rated strength. The static strength of swaged connections may be determined by calibrated analytical means or by representative tests with regard to size, material, and forming process. In doing so, due consideration shall be given to the variables of the connection configuration. These variables include, but should not be limited to

— the number, depth, and width of the grooves,

— the spacing between adjacent grooves,

— the radii of the re-entrant and protruding corners inside and between the grooves, and

— the pressure used in the swaging process.

The hoop stiffness of both tubular members shall be selected so that sufficient residual radial pressure remains at the contact points between the two parts, in order to develop adequate radial preload upon swaging.

15.2.9.2 Fatigue performance of swaged connections

The fatigue performance of swaged connections shall be demonstrated by tests representative of in-service actions, in which the definition of fatigue failure shall include cracking anywhere within the connection and unacceptable progressive slippage between the two tubular members.

15.2.9.3 Material for swaged connections

The material properties of the tubular members shall be selected to ensure that no cracks develop during the expansion of the tubular wall. In corrosive environments, such as offshore, the potential deleterious effects on the performance of the connection arising from general corrosion, stress corrosion cracking, corrosion fatigue, and crevice corrosion shall be considered.

For swaged connections of foundation piles, the fracture performance of the material shall remain within acceptable limits for the intended purpose following pile driving and swaging. Particular attention in this regard shall be given to the weld material present in the pile and the sleeve.

15.2.9.4 Installation of swaged connections

Swaged connections may be produced by mechanical or hydraulic means applied to the inside of the inner tubular member. In either case, the tool and the installation procedure shall ensure that the inner tubular member is swaged onto all of the grooves in the outer tubular member and that the intended amount of swaging of the tubular is reliably attained. One way to accomplish this is by conducting onshore trials and calibrating the tools.

15.3 Clamps for strengthening and repair

15.3.1 General

Subclause 15.3 applies to the design of clamps to strengthen or repair tubular members or joints, or to connect tubular members.

The common features among these clamps are that they are deployed in two or more pieces, are fastened by bolts, and typically surround a structural component, such as a joint or a member; the surrounded component is further denominated as the substrate component. However, a key distinction can be made on the basis of the function and effect of the bolts, resulting in two generic types of clamps: split sleeve clamps and prestressed clamps.

15.3.2 Split sleeve clamps

In split sleeves the initial tension applied to the bolts is merely intended to provide continuity of the sleeve parts. When sufficiently stiff and properly preloaded, the longitudinal joint can provide hoop stiffness continuity in the split sleeve. Otherwise, the bolts simply maintain the pieces together so as to produce an encasement around the substrate component. In both cases, the annulus or space created between the outer clamp pieces and the substrate component is filled with grout, but no prestress develops between the clamp and the substrate component.

15.3.3 Prestressed clamps

In prestressed clamps, the pre-tensioned bolts induce a continuous radial pressure on the substrate component, allowing friction on the contact surfaces, which then allows shear forces to be transferred between the parts of the clamp. The bolts can also provide a direct load path between the clamps parts. Three types of prestressed clamps are used, as follows.

a) **Prestressed grouted clamps** have the cavity left between the substrate members and the clamp filled with grout. After the grout is cured, prestress is applied normal to the grout surface to enhance the friction capacity strength. This prestress is normally achieved by stressing external bolts.

b) **Prestressed mechanical clamps** are similar to the prestressed grouted clamps, except that the grout is omitted and the clamp is in direct metal-to-metal contact with the substrate structure.

c) **Prestressed lined clamps** are the same as prestressed mechanical clamps, except that an elastomeric material is placed between the clamp and the substrate structure to accommodate local irregularities better.

CAUTION — Test data and in-service performance indicate that elastomeric lined clamps can be unreliable, see 15.3.6.3. Extreme care should be taken in their application.

15.3.4 Forces on clamps

15.3.4.1 Mechanism of force transfer

The internal forces in the substrate components and in the attached members generally translate into the following forces on the clamp:

a) shear forces and bending moments that tend to separate the clamp pieces;

b) axial force and torque along the clamp axis that tend to produce relative slippage between the clamp and the substrate members.

Prestressed clamps transfer forces by radial bearing contact and friction at the clamp to member interface. Therefore, the long-term bolt forces shall be of such magnitude as to ensure that the contact pressure induced by the bolts is not overcome by the separation action due to the forces applied to the clamp. At the same time, the contact pressure shall develop sufficient friction to resist slippage. Similar to grouted connections, the interaction of the bending moments and shear forces with the axial force, all acting at the free ends of the clamp, tends to enhance the slip strength of the clamp. Therefore, it is conservative to assume that the separation and slip forces do not interact.

15.3.4.2 Member forces

In determining the forces in the members of the structure, the effects of both the modified stiffness and the modified environmental actions, due to the presence of the clamp, shall be taken into account.

15.3.4.3 Bolt forces

To evaluate the bolt forces necessary to resist the separation and slip actions on the clamp, the contributions from all actions on the clamped members (the substrate members) and the members attached to the clamp body shall be taken into account. The forces induced in the bolts may be determined by accepted analytical or numerical engineering methods or by testing. Particular attention shall be paid to the forces in the bolts at the ends of the clamp. Approximate methods for calculating the forces induced in the bolts by the separation forces and by the acting slip force at the clamp interface are presented in A.15.3.4.3.

15.3.5 Clamp design

15.3.5.1 General approach

Clamps shall be designed to fully sustain and transfer all of the forces and moments acting in:

a) members attached to the clamp;

b) substrate members on which the clamp is installed.

However, in cases where adequate static or fatigue strength of the intact or damaged substrate structural component can be demonstrated, the clamp may be designed to transfer only the portion of the forces and moments attracted by the clamp.

The design process involves selecting the size, number, distribution and preload of the bolts, in conjunction with a clamp configuration such that the clamp shall sustain and transfer the forces in the clamped and attached members without opening, slipping or crushing the clamped members. If time-varying fatigue forces are acting on the clamp, the fatigue performance of both the clamp and the bolts shall also be evaluated.

15.3.5.2 Check of the clamped member

Because the bolt forces are directly resisted by hoop forces in the clamped tubular members, the substrate members shall be checked to avoid crushing at the free end of the clamp. For mechanical and lined clamps, crimping (local buckling of the part of the substrate member not contained within the clamp) shall also be checked, if appropriate. In prestressed grouted clamps, advantage may be taken of the strength of the grout annulus to carry compression forces when calculating the radial crushing pressure on the clamped member, provided that the grout annulus is circumferentially continuous. Crushing of the clamped member can be alleviated by lengthening the clamp or filling the member with grout.

15.3.5.3 Static design of bolts

The long-term bolt tension shall be determined as the largest of the bolt forces required to

a) maintain a positive contact between the clamp and the substrate member under the action of the separating forces from factored actions, and

b) resist slippage under the action of the separating forces from factored actions.

Bolts shall be sized to sustain the long-term tension in accordance with the requirements given in 15.2.8.5 for threaded fasteners.

The contribution from individual bolts to the overall strength of the clamp shall be considered at the design stage to ensure that failure of any individual bolt or loss of tension in one or more bolts does not lead to overall failure of the clamp. The degree of bolt redundancy should be linked to the extent of inspection (see 15.2.8.4 for threaded fasteners).

15.3.5.4 Fatigue design of bolts

The evaluation of the variable bolt stresses caused by the time-varying forces shall be conducted by FEA or by any other demonstrably conservative engineering method (see A.15.3.5.4)

Fatigue lives may be predicted using the cumulative Palmgren-Miner's damage ratio with an appropriate S–N curve. In selecting the S–N curve, account shall be taken of the type of bolts, the diameter and the exposure conditions (see A.15.2.8.6 for threaded fasteners).

15.3.5.5 Interface transfer strength of prestressed clamps

The ability of prestressed clamps to transfer force at the clamp-substrate interface depends on numerous variables. These include the bolt size and the level of preload, the number and distribution of the bolts, the length and stiffness of the clamp, the size and surface finish of the clamped member, and, if applicable, the grout strength. In the absence of specific strength data, the slip strength of prestressed clamps may be calculated using the expressions given in A.15.3.5.5.

When calculating the slip strength of a prestressed clamp, the loss of bolt tension from transfer losses upon tensioning and long-term effects on the grout and steel shall be allowed for in the derivation of the long-term bolt forces (see 15.2.8 for threaded fasteners).

In prestressed clamps comprising a continuous ring of grout, a proportion of the bolt preload is resisted by hoop compression in the grout, leading to a reduction of the effective prestress at the grout/member interface. This loss of effective prestress shall be considered in the slip strength calculations.

15.3.5.6 Interface transfer strength of split sleeve clamps

The interface transfer strength of split sleeve clamps may be taken as that given for grouted connections (see 15.1.6), provided that the hoop stiffness continuity of the longitudinal seams can be demonstrated by showing that no separation will occur on any part of the contact faces along the splits under any combination of forces and allowing for all losses of bolt stress, including time dependent effects. Where hoop stiffness continuity cannot be demonstrated, the strength of the split sleeve clamp is reduced, while the degree of reduction depends on the contribution of the sleeve size to hoop stiffness, the configuration of the seam flanges, and the bolting arrangement. Again, tests are the only reliable approach to quantifying the strength of this type of clamp (see 7.7).

15.3.6 General requirements for bolted clamps

15.3.6.1 Mechanical clamps

Reliance on metal-to-metal contact to develop the interface transfer strength in mechanical clamps requires that due account shall be taken of

a) fabrication and installation tolerances, and

b) the finish of the contacting surfaces (see A.15.3.6.1).

15.3.6.2 Grouted clamps

Clamps using grout as a filler medium provide greater flexibility for installation tolerances than mechanical clamps. The following additional points shall be considered when using grouted clamps:

a) the completeness of grout filling between the clamp and the substrate member, which can be affected by the position of grout inlets and outlets, the effectiveness of seals, and grout bleed;

b) the interface transfer strength, calculated in accordance with Equations (15.1-4), (15.1-5) and (15.1-6), provided the clamp body can be regarded as continuous;

c) the shrinkage and creep of grout, effectively reducing the contact pressure between the clamp and the clamped member;

q) the relative movement between the clamp and the substrate member during grout curing and prior to bolt force application (see 15.1.5.3);

r) the requirements of 19.6 in respect of grout materials.

15.3.6.3 Lined clamps

Elastomeric lined clamps should not be used where stiffness is important (see A.15.3.6.3). Relative stiffness is particularly important for partial strength repairs, where the original damaged structure can be significantly stiffer than the repair or where the repair relies to any extent on the stiffness contribution from the existing structure through the load path being repaired.

Elastomeric lined clamps should not be used for primary structural members unless specific data on strength and long-term effects are available.

15.3.6.4 Corrosion protection

Adequate corrosion protection shall be provided when attaching clamps and additional strengthening of members to an existing structure. Underwater corrosion protection is preferably achieved by providing the clamp with sacrificial anodes or, alternatively, by ensuring electrical continuity between the clamp and the structure, if appropriate. In either case, continuity between the clamp body and the bolts shall be provided to ensure protection of the bolts. The use of coated bolts, whether these are cathodically protected or not, is recommended.

It is also important to consider the interaction between any repairs and the existing structure with regard to levels of cathodic protection, and to any possible damaging effects on the repair components, especially the bolts.

15.3.7 Bolting considerations

The requirements of 15.2.8 on threaded fasteners apply.

16 Fatigue

16.1 General

16.1.1 Applicability

Clause 16 presents general considerations and methods for verifying that fixed steel offshore structures satisfy the fatigue limit state (FLS) at all locations in the structure during their entire life, from fabrication via the in-place situation to the removal of the structure.

16.1.2 The fatigue process

Fatigue refers to the cumulative damage done by repeated application of time-varying stresses at a specific location in the structure. These time-varying stresses are caused by variable actions, especially, but not exclusively, due to wave action. For general reference, a brief overview of the fatigue process is given in A.16.1.2.

16.1.3 Fatigue assessment by analysis using S–N data

Normally, during design a fatigue assessment is performed by analysis using S–N data. The following major aspects for determining fatigue damage due to wave action are addressed in Clause 16:

a) general requirements for a fatigue assessment (see 16.2);

b) a description of the long-term wave environment during the design service life (see 16.3);

c) a description of particular aspects of the stress analyses needed for a fatigue assessment (see 16.4);

d) the characterization of the stress range data governing fatigue (see 16.5);

e) the long-term local stress range history (see 16.6 to 16.9);

f) the geometric stress range concept, including stress concentration factors (see 16.10);

g) the fatigue resistance of the material (see 16.11);

h) the fatigue damage assessment during a period of exposure (see 16.12).

Points e) to h) are specific to the particular location in the structure being considered and shall be repeated for all locations that are fatigue sensitive.

The various aspects a) to h) are integral parts of a fatigue assessment and cannot be seen as independent elements. Despite the fact that they generally involve distinctly different technologies, which are often developed by different specialists, they shall be treated in an interrelated manner. This is essential in view of the large influence they can jointly have on the results obtained. Lack of appreciation of these interrelationships presents an inherent danger to any fatigue assessment. However, the various steps in the assessment are usually performed separately and in sequence; therefore it is most convenient to also discuss them separately.

Clause 16 provides the fundamental requirements that shall be met to achieve reliable results. It leaves flexibility where possible or necessary to accommodate differences in the details of calculation procedures that are embodied in established practices or in computer programs that are used in the industry.

The design assessment is concluded with some observations on fatigue damage due to causes other than wave action (16.13) and some practical design considerations (16.14).

16.1.4 Fatigue assessment by analysis using fracture mechanics methods

In some special cases, fatigue lives can be determined using fracture mechanics methods (see 16.15). Fracture mechanics methods can also be used for special purposes associated with, but not directly aimed at, designing components of a new structure, and for determining the remaining fatigue life of components in which a crack has developed.

16.1.5 Fatigue assessment by other methods

Normally, a fatigue assessment requires detailed analyses using rational methods. However, when it can be demonstrated that either the fatigue limit state will not be reached in the design service life or, alternatively, when relevant prior experience exists to reliably indicate that this will be the case, a detailed fatigue analysis as per 16.1.3 or 16.1.4 may be replaced by a simpler assessment(s).

16.1.6 Fatigue assessment of existing components

Fatigue assessments of components in existing structures may be based on fracture mechanics methods or on a careful evaluation of inspection results, in lieu of a detailed fatigue analysis using the long-term stress range history and S–N curves. Additionally, fatigue lives of existing components can be improved by a number of weld improvement methods; see 16.16.

16.2 General requirements

16.2.1 Applicability

The requirements given in this International Standard apply specifically to fixed steel structures of welded tubular space frame configuration. However, the principles described are much more widely applicable to steel structures in general. This includes structure types such as free-standing or braced caissons, compliant bottom founded structures, jack-ups and topside structures. Assessing the degree of applicability, or the need for specialization of the requirements for a particular type of structure, is the responsibility of the user of this International Standard.

When used for components of non-tubular configuration, or for framed structures containing such components [e.g. components constructed of welded, rolled or fabricated beams, or of welded (un)stiffened plates], care shall be exercised to adapt the procedure for selecting the most appropriate stress parameter that governs the fatigue process in these cases.

All components of a structure are potentially sensitive to fatigue damage accumulation. In particular, every welded connection, every connection joined by other means than welding, every attachment, every structural discontinuity, and every place where some form of stress concentration is present is a potential location of fatigue cracking and can require individual consideration. It is therefore often useful to first perform lower bound screening assessments to determine those locations in the structure that are most susceptible to fatigue damage accumulation and that are to be subsequently assessed in detail on an individual basis.

The requirements in this International Standard are applicable to the fatigue design of new structures as well as the fatigue assessment of existing structures. However, they only relate to fatigue evaluations of "uncracked" locations; therefore, in the case of existing structures, the proviso is that there be no crack already present. The reason for this is that the local stress situation at a crack is not captured by the stress calculations described herein, while the fatigue crack growth process is neither adequately modelled by the S–N curves describing the fatigue resistance of the material or the structural detail. An assessment of the acceptability of a pre-existing crack requires a fatigue crack growth evaluation using fracture mechanics methods; see 16.15.

In this context "uncracked" refers to the absence of macroscopic cracks of a size that would be detectable by suitable inspection techniques. Microscopic defects of a size that cannot be detected by suitable inspection techniques are always present in welded connections, and these are not considered as cracks in the present sense. Successful application of the requirements given in this International Standard to the fatigue design of new structures implies that the structure at the end of its design service life will not have cracks of critical size.

Requirements for fatigue design may be relaxed when they are complemented with requirements for in-service inspection to verify actual fatigue performance; see 16.14.8 and A.16.14.8.

16.2.2 Fatigue crack initiation and crack propagation

The small crack-like defects present in welds, or the material in the heat affected zones adjacent to welds, will act as initiators for crack growth under the influence of variable stresses. Therefore, for the procedures described in this International Standard, there is only a crack propagation phase and no crack initiation phase in welded construction.

16.2.3 Sources of variable stresses causing fatigue

The predominant source of variable stresses causing fatigue in an offshore structure is wave action in the in-place situation during the design service life. However, other sources of variable stresses causing fatigue damage also require consideration; see 16.13. The observations and requirements given in 16.3 to 16.12 largely refer to the in-place situation, but with due alteration of details they are much more widely applicable.

16.2.4 Service life and fatigue life

Design service life is defined in ISO 19900 as the assumed period during which a structure is to be used for its intended purpose with anticipated maintenance, but without substantial repair being necessary. For a fixed offshore structure this is the period during which it is in-place at its offshore location. The design service life is preceded by a pre-service period, which generally comprises a period of onshore fabrication in the yard, a period of ocean transportation and a period of (offshore) installation.

The structure's design service life shall be specified by the owner — where appropriate, in accordance with the requirements of a regulator where one exists. For the purpose of a fatigue assessment, the design service life shall not be taken as less than 10 years. The fatigue life at any location in the structure, calculated in accordance with 16.12, shall not be less than the design service life.

Fatigue damage incurred prior to the design service life, and fatigue damage due to causes other than wave action in the in-place situation, shall be taken into account; see 16.12 and 16.13.

16.2.5 The nature of fatigue damage

Fatigue damage is a highly localized phenomenon; the damage originates at one or several neighbouring locations in the structure and develops from there. The methods described in this International Standard do not deal with progressive fatigue failures, which can eventually lead to the global failure of a structure. Therefore, it is meaningless to speak of fatigue of a structure in a global sense. Consequently, the local situation with respect to stress variations, and the local details of construction and workmanship during fabrication, have a profound effect on the fatigue damage done.

All these features have to be taken into account in a fatigue assessment as accurately as possible if the results are to be reliable. Therefore, a fatigue analysis generally requires much more detailed models of the actions and of the structure, as well as much more refined analysis procedures than those of a strength analysis.

16.2.6 Characterization of the stress range data governing fatigue

Due to the highly localized nature of fatigue damage, it is very important to select appropriate stress range parameter(s) for describing the local stress range history as discussed in 16.5; see also A.16.1.2.

16.2.7 The long-term stress range history

The stress variations occurring during a particular period at a specified location in the structure shall be specified in the form of a long-term stress range history. The period considered can, for example, be the design service life for the in-place situation, or the duration of yard fabrication or the duration of an ocean tow for the pre-service situation.

The long-term stress range history can be given as a function of time or as a statistical distribution. Specification as a function of time is usually impractical and, more importantly, for nearly all cases of practical interest, the sequence of events producing the stress range time-history during the period considered is, *a priori*, unknown. A selected time-history would hence merely be an arbitrary choice from an infinite number of equally probable possibilities. Experience further indicates that the impact of the sequence of events on the assessment of fatigue damage averages out with time. When considered over a long period, as is normal for a fatigue assessment, the effect is hence very small if noticeable at all. Therefore, the long-term stress range history is normally specified in the form of a statistical distribution.

The long-term stress range history can be determined by means of the following:

- a series of spectral analyses if a spectral fatigue analysis method is used, as described in 16.7;
- a series of deterministic analyses if a deterministic fatigue analysis is used, as described in 16.8;
- other approximate procedures that are usually based on previous experience for the type of structure and the geographical area concerned, see 16.9.

Guidance on the choice between a spectral and a deterministic fatigue analysis is given in 16.6.

The use of other approximate procedures can only be justified for screening analyses during initial design phases or if previous experience indicates that fatigue is not an important design issue. Even in these cases, great care combined with sound knowledge shall be exercised.

16.2.8 Partial action and resistance factors

In determining stress variations for a fatigue analysis the partial action factors shall be taken as 1,0. The partial resistance factor on the fatigue resistance shall also be taken as 1,0. Safety against fatigue failure is provided by using fatigue resistance in the form of a design S–N curve, which is the mean minus two standard deviations of log N. Overall safety against failures associated with fatigue damage accumulation is further provided by an additional fatigue damage design factor $\gamma_{FD} \geqslant 1{,}0$. Local experience can be taken into account by a local experience factor k_{LE}, which can be larger or smaller than 1,0; see 16.12.3.

16.2.9 Fatigue resistance

The fatigue resistance of the material in or around a weld to variable stresses is given in the form of empirical S–N curves, where S is a constant amplitude stress range and N the number of cycles to failure. Many S–N curves have been determined and can be found in the literature. Each curve is strictly associated with a particular definition of S and N, as well as a particular structural detail; they shall not be used with variables that are defined in a different manner or for different structural details; see also A.16.1.2. The S–N curves that shall be used are presented in 16.11.

16.2.10 Fatigue damage calculation

The long-term stress range history and the fatigue resistance both relate to a particular location in the structure. These two items are combined via the Palmgren-Miner rule to determine the fatigue damage at the location during the period under consideration; see also A.16.1.2. Application of the Palmgren-Miner rule is discussed in 16.12.

16.2.11 Weld improvement techniques

Weld improvement techniques shall not be used for the design of a new structure in lieu of reducing fatigue damage by means of appropriate modifications to the design, as long as such modifications are still practically possible. Appropriate design modifications can include

- — reducing the variable actions causing stress variations, if possible,
- — changing the geometry or the dimensions of the component(s) involved to reduce the magnitudes of the stress variations (the action effects) occurring, and
- — changing the construction details of the component(s) concerned.

Weld improvement techniques (see 16.16) may be considered as a last resort measure that can only be justified when the project is too far advanced to still enable design changes to be introduced in a timely and practical manner.

16.3 Description of the long-term wave environment

16.3.1 General

In a description of the wave environment, short-term and long-term conditions are distinguished.

Short-term conditions refer to a stationary situation in which the sea remains the same in a statistical sense. Such stationary situations are limited in duration and are normally assumed to last for a standard period of 3 h. A stationary situation (short-term condition) of the sea is referred to as a sea state. A short-term condition of the sea can be realistically described by a theoretical model in the form of a wave spectrum, the parameters of which can be chosen to reflect the actual wave conditions at a particular time and location.

Long-term conditions refer by definition to situations in which the wave environment is no longer stationary but changes. Long-term wave conditions hence always consist of a succession of sea states. As there are no theoretical models to describe the long-term wave climate, long-term conditions at a particular location are always empirical.

Long-term conditions typically refer to the duration of a season, a year or the design service life of a structure. For a fatigue assessment of the in-place situation, the long-term conditions during the design service life or the long-term conditions during a typical year can be used. In the latter case, it is assumed that the conditions in a typical year repeat themselves every year during the design service life. For a fatigue assessment of pre-service situations, the long-term conditions relate to the duration of yard fabrication, the duration of the transportation from the yard to the offshore location, or the duration of the offshore installation.

16.3.2 Wave scatter diagram

The most suitable representation of the long-term wave environment at a location is the wave scatter diagram. Ideally, a wave scatter diagram specifies the probability density function, $p(H_S,T_R,\theta_m)$, of the joint occurrence of the three main parameters defining a sea state, being the significant wave height H_S, a representative period, T_R, and the mean wave direction, θ_m. However, joint statistics of these three parameters are rare and the best that is usually available is a two-dimensional wave scatter diagram, $p(H_S,T_R)$.

In some cases, the available data permit the determination of directional wave scatter diagrams for eight, or preferably 12 equally spaced directions. Directional wave scatter diagrams are discrete subsets, $p(H_S,T_R)$, of the three-dimensional wave scatter diagram, $p(H_S,T_R,\theta_m)$, for specific wave directions.

Wave scatter diagrams usually represent the long-term wave environment during a (typical) year and should be based on several years of site-specific data in order to ensure that they adequately represent the wave environment at the location of the structure. The data can be determined by measurements, by hindcast modelling or by a combination of the two.

16.3.3 Mean wave directions

Where no information on wave directions is available, an omnidirectional wave scatter diagram can be used that does not distinguish the mean direction of individual sea states. It is then assumed that the three-dimensional wave scatter diagram, $p(H_S,T_R,\theta_m)$, can be transformed into an omnidirectional two-dimensional wave scatter diagram, $p(H_S,T_R)$, by a separation of variables:

$$p(H_S,T_R,\theta_m) = p(\theta_m)\,p(H_S,T_R) \tag{16.3-1}$$

where $p(\theta_m)$ is the probability of occurrence of each mean wave direction irrespective of the significant wave height H_S and the representative period T_R.

Mean wave directions are difficult to measure and information on them is not often available. In such cases, the statistics of wind directions can be used as an approximation to the probability of occurrence of mean wave directions. It is then assumed that the distribution of the mean wave direction $p(\theta_m)$ coincides with the distribution of the mean wind direction $p(\theta_w)$.

Where applicable, care shall be taken to account for special geographic features, such as restrictions in fetch from certain directions or special sea floor topography, which can impose limitations on wave generation for specific wind directions.

16.3.4 Wave frequency spectra

Each sea state in the wave scatter diagram is described by a two-parameter wave spectrum. For fatigue analysis purposes, a wave spectrum shape shall be chosen that is representative of the average distribution of wave energy over wave frequency for a large number of sea states. For fully developed sea states, the two-parameter Pierson-Moskowitz spectral formulation is normally most appropriate. For sea states that are not fully developed, the two-parameter Jonswap wave spectral formulation can be preferred.

The wave period used in the spectral formulation shall be consistent with the representative period, T_R, for the data in the wave scatter diagram.

16.3.5 Wave directional spreading function

The water surface elevation in a real sea is short-crested (see A.16.3.1), i.e. the frequency components are spread over a range of directions around the mean wave direction. To accommodate the effect of wave spreading on structural responses, a directional spreading function may be used where appropriate.

16.3.6 Periodic waves

In all fatigue analysis methods, the basic stress variations in the structure are calculated using individual periodic waves. The particular way in which the periodic waves are used in the various fatigue analysis methods is described in 16.7 to 16.9.

16.3.7 Long-term distribution of individual wave heights

In a deterministic fatigue analysis, the long-term wave environment is specified by the long-term distribution of individual wave heights, which can be derived from the wave scatter diagram; see A.16.3.7.

16.3.8 Current

Currents are normally not taken into account in calculating applied wave actions for a fatigue analysis (see A.16.3.8). However, fatigue damage due to vortex induced vibrations (VIV) caused by currents shall be considered; see 16.13.

16.3.9 Wind

For a fixed offshore structure, variable actions due to wind are small in comparison with variable actions due to waves and do not need to be considered for the in-place fatigue analysis.

There is no direct wave action on the topsides of a fixed offshore structure, and therefore variable actions due to wind are normally the most important source of fatigue stresses in components of the topsides structure in the in-place situation, particularly for long slender structures such as flare structures.

Wind-induced fatigue shall be considered for other situations than the in-place situation; see 16.13.

16.3.10 Water depth

In a fatigue analysis of the in-place situation the average water depth during the design service life of the structure shall be used; the potential effect of subsidence should be considered, see A.16.3.10.

16.3.11 Marine growth

For new structures, the design marine growth thickness shall be used.

16.4 Performing the global stress analyses

16.4.1 General

General guidance on structural modelling and analysis is given in Clause 12. The guidance in 16.4 relates specifically to stress analyses for a fatigue assessment.

All methods of fatigue assessment by analysis are based on the results of a large series of global stress analyses in individual periodic waves (see 16.3.6). The calculated stress variations are the input data for all subsequent steps in the fatigue assessment and should therefore be fully reliable. The computer models used for the stress analyses shall be suitably detailed to ensure that the nominal stress variations at a particular location in the structure are adequately captured and are compatible with the purpose and the chosen method of fatigue analysis. This applies to both the action model and to the structural model. Neither model used for the strength analysis of the structure is normally detailed enough for stress analyses for fatigue assessment purposes and both require suitable additions and refinements.

For the action model, only time-varying direct actions are required; permanent actions do not need to be considered.

The structural model shall include all relevant characteristics of the structure and its foundation, in order to correctly reflect the magnitude and distribution of its stiffness. Where dynamic structural response is involved, the magnitude and distribution of the structure's mass shall also be correctly represented. Care shall further be taken to model the geometry of the structure in sufficient detail to capture any asymmetry in the structure's stiffness, in its mass or in the applied actions in order to ensure that torsional responses are also properly taken into account. Where required, damping can normally be modelled by viscous damping using a damping ratio (a non-dimensional damping coefficient).

For space frame structures the time-varying stresses in an individual structural component are due to overall frame action and to direct wave action on and around the component considered. Direct wave action usually becomes more important the higher the component's elevation in the structure. Therefore, all relevant members in the (near) vertical frames and in the plan framing levels shall normally be included and shall be modelled in a realistic manner. Recognizing the potential influence of asymmetry, the lack of symmetry in bracing patterns, with respect to the overall configuration of the structure in combination with the lack of symmetry in applied actions for seemingly similar wave directions, shall not be overlooked. The transfer of applied actions to foundation piles, any foundation constraint provided by conductors, where present, and associated details in the modelling of the plan framing at the sea floor, shall also be included in the model, in order to ensure that the distribution of support reactions through the structure is correctly represented for different levels of the applied actions.

In many respects, modelling particulars for the different fatigue assessment methods are the same. Those aspects that are common are described here. Aspects that are specific to one of the methods are described in 16.7 to 16.9.

16.4.2 Actions caused by waves

The actions on the structure for the stress analyses are the direct actions caused by periodic waves. The basis of the procedure for calculating these actions is described in 9.5, with the following specializations and modifications :

a) current and the current blockage factor shall not be used (see 16.3.8);

b) instead of the storm water depth, the water depth specified in 16.3.10 shall be used;

c) in the context of the procedure used in 9.5, the wave directional spreading factor shall be 1,0;

d) care shall be taken that the influence of marine growth on the stresses calculated for fatigue analyses is correctly taken into account, as marine growth affects both the drag and the inertia term in Morison's equation (see below);

e) for fatigue analyses, the values of the drag and inertia coefficients shall not be taken from 9.5.2.3 (see below);

f) the conductor shielding factor shall be 1,0.

For spectral and deterministic fatigue analysis methods, different periodic wave theories can be applicable; see 16.7 and 16.8.

All fatigue assessment methods are based on stress ranges. Each stress range in an individual periodic wave is due to a full wave cycle. The resulting stress cycle at a particular location consists of contributions from an axial and a bending stress cycle, which are in general not synchronous. To determine the appropriate resulting stress range in the time domain, a full cycle of each periodic wave shall be stepped past the structure, requiring a substantial number of separate wave positions with the associated calculations (see A.16.4.2). When the whole formulation for the structural analysis is adequately linearized, the stress calculations may alternatively be performed in the frequency domain. In this latter case, two calculations for each periodic wave suffice, one for the real (in-phase) and one for the imaginary (out-of-phase) components of the applied wave actions.

For all fatigue assessment methods, the actions due to waves shall be calculated for a sufficient number of wave directions to adequately capture the influence of wave direction on local stress variations resulting both from overall frame action due to global hydrodynamic actions and from direct wave action on individual components. For typical configurations of steel structures a minimum of 8 to 12 wave directions covering the full 360° of the compass are required for a comprehensive representation of the stresses at a particular location. The selected wave directions should normally coincide with the mean wave directions for which statistical information in the wave scatter diagram is available (see 16.3.2).

The model for hydrodynamic actions shall include a three-dimensional representation of all components of the structure that are exposed to wave action. Whereas for a global strength analysis horizontal actions are usually the most important, for a fatigue analysis careful attention shall be paid to the accurate modelling of actions on components in all directions. This notably includes vertical applied actions on submerged plan frames.

Appurtenances subjected to wave action shall be included and shall be properly modelled in their correct location and with their appropriate connection details, so as to realistically represent the transfer of any actions on them to the structure. Appurtenances that shall be modelled in this manner include, but are not limited to conductors, caissons, risers, J-tubes, boat landings, fenders, launch rails, pile guides, pile sleeve assemblies, etc. Miscellaneous attachments such as anodes also attract hydrodynamic actions, which shall be included as well. However, wave actions on anodes are representative of a more distributed type of action and may be modelled with an increase in the drag and/or inertia coefficients in the Morison equation.

The influence of marine growth shall be included by taking into account the influence of an increase in the diameter on the drag term as well as on the inertia term in Morison's equation. The drag coefficient shall additionally reflect the influence of roughness. All this is normally achieved by using an effective hydrodynamic diameter in conjunction with appropriately adjusted drag and inertia coefficients.

The analyses shall be run for a wide range of wave heights and periods, the majority of these being much lower and shorter than the design wave height and period for the strength analysis. The inertia term in Morison's equation is hence much more important in fatigue analyses than for the strength analysis in an extreme storm condition, up to the point that the inertia term can become dominant over the drag term. Therefore, the cross-sectional area (or the equivalent volume of components with shapes that are not tubular) and the associated choice of inertia coefficients require careful attention.

As short wave periods are associated with short wave lengths, it is generally necessary to calculate the applied actions at a larger than usual number of discrete points along the length of components for extreme wave analyses, in order to adequately define the distribution of wave actions over individual components as well as the total wave action on the structure as a whole. Only in this manner can the influence of distributed actions on member end moments be properly taken into account.

In fatigue analyses, an inertia coefficient of $C_m = 2{,}0$ shall be used for tubular members (see A.16.4.2). The drag coefficient C_d may be chosen in accordance with A.9.5.2.3. The coefficients that shall be used for non-tubular members and non-tubular appurtenances are subject to special consideration.

16.4.3 Quasi-static analyses

As the actions on the structure are by definition time-varying, the stress responses are also time-varying. Structures for which dynamic behaviour is significant are referred to as *dynamically responding structures* (see 12.5.2). When the influence of structural accelerations can be neglected, stresses may be calculated using a quasi-static linear structural analysis (see 12.5.3). The acceptability of neglecting dynamic effects should be carefully reviewed; some guidance is provided in 12.5.2 and A.16.4.3.

16.4.4 Dynamic analyses

16.4.4.1 General

In global structural analyses to determine member nominal stresses, the steady state dynamic response to each of the periodic waves stepped past the structure shall be included by performing a dynamic linear analysis (see 12.5.5). The influence of dynamic response on the long-term local stress range history shall be determined using spectral analysis and a frequency domain dynamic solution.

The position of the platform natural frequencies relative to the peaks and troughs of the transfer function of the applied wave action can have a profound effect on the level of dynamic response. Since the natural frequencies can vary significantly, depending upon design assumptions and operational deck mass, the theoretical natural frequencies shall be reviewed and their position adjusted if necessary (see A.16.4.4.1).

16.4.4.2 Mass

The mass model shall include all structural elements, conductors, appurtenances, miscellaneous attachments, grout, marine growth, entrapped water and added mass. A lumped mass model is sufficient to obtain global structural response. However, it does not always adequately predict local dynamic response. Where necessary, local responses shall be examined using a consistent mass model.

The model of the topsides shall represent the total topsides mass, the centre of mass and the rotational inertia of the deck topsides mass in order to correctly account for torsional response.

16.4.4.3 Stiffness

The structural model shall include an appropriate three-dimensional distribution of structure stiffness. The stiffness of the deck structure shall be modelled in sufficient detail to adequately represent its effect on the behaviour of the structure due to fatigue excitation.

Foundation springs shall be included for all foundation components that carry significant reactions (e.g. main piles or pile groups, skirt piles and conductors); the relative stiffness of different components shall be taken into account.

The foundation stiffness shall be modelled by an equivalent linear representation, such that the foundation deflections and rotations in a sea state representing wave conditions that contribute significantly to fatigue damage are correctly reflected. The centre of fatigue damage sea state, calculated as described in A.16.7.2.3, is appropriate for this purpose.

The foundation stiffness can have a large effect on the natural period(s) of the structure. When assessing the range of natural period values which can occur, upper and lower bound foundation stiffness values shall be considered.

16.4.4.4 Damping

Energy dissipation due to damping has a profound effect on structural response calculations at and in the immediate vicinity of the resonant frequency or frequencies. Outside the resonance region, the influence of damping is negligible.

Offshore structures are very lightly damped. The physical sources of damping are difficult to determine and to quantify with any certainty. For the purpose of a fatigue analysis damping shall be modelled by a viscous damping coefficient that accounts for all sources of damping including structural, foundation and hydrodynamic effects. For typical pile founded tubular space frame structures, a total damping value of 1 % to 2 % of critical is appropriate. For other structural configurations and for topsides structures, damping values are typically of similar magnitude but should be selected with care.

16.5 Characterization of the stress range data governing fatigue

The general conditions of a fatigue assessment are discussed in 16.1.2 and A.16.1.2. The requirements in 16.5 to 16.12 are specifically concerned with the fatigue assessment of welded connections of tubular and non-tubular components in a fixed steel offshore structure caused by variable actions due to waves during the structure's design service life. However, the concepts discussed are more widely applicable to fatigue assessment in general.

Residual stresses in or around welds can be assumed to have magnitudes equal to the stress at which the material yields in tension. Stress variations in or around welds can hence be assumed to always range downwards from the yield strength in tension. Consequently, other than for plain steel, the mean stress and the stress ratio, R (the ratio of the minimum to the maximum stress during a cycle), of the applied stresses are inappropriate parameters to assess fatigue damage potential in welded components. For applied stresses that are less than or equal to half the yield strength, the local R value will always be greater than or equal to zero.

Therefore, the stress range is universally accepted as the sole stress parameter that governs fatigue of welded components.

The most important factors influencing fatigue damage are the stress range at a location, the number of applied cycles of a particular stress range magnitude, and the fatigue resistance of the material.

A global analysis model is normally used to determine nominal stress ranges in the vicinity of the connection. Requirements for performing global analyses are given in Clause 12 and in 16.4. These analyses shall provide the full cycle of relevant stress components (e.g. axial stresses and bending stresses), which are generally not synchronous; for harmonic stresses, this manifests itself in phase differences. The non-synchronous nature (phase differences) between stress components shall be duly taken into account.

For beam elements, nominal stresses are calculated at member ends. The relevant stresses are the stresses that are parallel to the longitudinal axis of the beam, i.e. axial stresses, in-plane bending stresses and out-of-plane bending stresses. The stress variations of these three nominal stress components shall be kept separate and shall not be combined. Shear stresses and torsional stresses may be neglected. For elements other than beam elements, special considerations apply for determining which stress component(s) are to be considered.

For tubular connections, geometric stress ranges (GSRs) are determined at specific locations of the connection by multiplying the nominal stresses first by stress concentration factors (SCFs) and then combining the stress components as described in A.16.5. The SCFs are described in 16.10.2.

For non-tubular connections, GSRs are determined at specific locations of the connection by assigning a joint classification to the connection (or to a particular construction detail of the connection) and specifying which stress range is to be used as the GSR (see 16.10.3).

The GSR is subsequently used to determine the long-term local stress range history, which is ultimately combined with the corresponding S–N curve (see 16.11) in performing the fatigue assessment (see 16.12).

Both test data and analytical techniques may be used to determine an SCF for the specific location at the tubular joint and the type of stress considered, in relation to the corresponding S–N curve, provided that it can be demonstrated that the magnitude of the SCF can be estimated reliably with the selected method. Care shall be taken when using results obtained from limited test programmes with models that are not entirely adequate for the purpose. Similar care shall be exercised when using results from analytical investigations performed with techniques that are not entirely adequate for modelling the local situation.

16.6 The long-term local stress range history

16.6.1 General

The long-term local stress range history is a statistical distribution that combines information on the magnitudes and numbers of cycles of the stress ranges at a specific point during a given period of time. It thus provides the number of occurrences, n_i, of local stress ranges, S_i, where S_i is the GSR (see 16.5). All stress ranges that can potentially contribute to fatigue damage shall be included in the distribution.

For the in-place situation, the stress ranges are caused by wave action. The long-term distribution is then determined by summing the weighted short-term distributions for all individual sea states and wave approach directions occurring during the design service life. The weight factor for the contribution of each short-term distribution is the product of the probability of occurrence of each sea state and the probability of occurrence of its wave approach direction.

The GSR stress data from which the short-term distribution of S_i for each individual sea state is compiled can be obtained from

a) a probabilistic determination using spectral analysis methods, or

b) a deterministic determination using individual periodic waves.

The two methods require different computational effort and provide different levels of accuracy. Their main fundamental difference lies in the manner in which the long-term wave climate is represented, with corresponding differences in the subsequent steps; see 16.6.2 and 16.6.3.

16.6.2 Probabilistic determination using spectral analysis methods

In the spectral analysis method, a sea state (the short-term wave environment) is represented by a two-parameter wave frequency spectrum (see 16.3.4). This spectral description provides the most comprehensive representation of the features of waves in a real sea. The description retains the random nature as well as the frequency content of a real sea. As a result of this it is able to realistically model the effect of wave frequency on applied wave actions and structural response. Thus, the spectral procedure

— enables a probabilistic determination of the short-term distribution of S_i for each sea state and at each location of interest in the structure, and

— is the only method that can be applied to both (quasi-)statically responding and dynamically responding structures.

A disadvantage of the spectral method is that the global analyses by which the nominal stresses are determined require linearization of the hydrodynamic drag action, of wave inundation effects around the still water level and of foundation behaviour. Appropriate treatment of these aspects shall be carefully considered in all applications. Application of the spectral method is detailed in 16.7.

The spectral method is the preferred method for the final fatigue assessment of all structures, unless it can be reliably demonstrated that another method can be justified.

16.6.3 Deterministic determination using individual periodic waves

Instead of a probabilistic description using wave spectra, the ocean environment is sometimes described by a series of deterministic individual waves. These are periodic (regular) waves with a particular height, period and direction, and an associated number of occurrences. Such periodic waves are merely an abstraction of reality for analysis purposes and do not attempt to produce a realistic representation of the features of waves in a real sea. Consequently, they reflect neither the random nature nor the frequency content of real waves. Hence, the deterministic procedure

— produces a deterministic short-term distribution of S_i for each sea state and at each location of interest in the structure, and

— is not suitable for application to dynamically responding structures.

Guidance on whether the dynamic behaviour of a structure needs to be taken into account or may be neglected is given in 16.4.3.

The deterministic method does not require linearization of the global analyses by which the nominal stresses are determined. Application of the deterministic method is detailed in 16.8.

Where a deterministic method can be justified, it may be used for screening evaluations during the initial design phases and for the final fatigue assessment of structures that are not critically fatigue sensitive.

16.6.4 Approximate determination using simplified methods

Experience from many analyses as well as some measurements from existing structures have shown that in many cases the long-term distribution of local stress ranges can be represented reasonably well by a Weibull distribution or combination of two Weibull distributions.

In extratropical areas (outside the areas where tropical cyclones can occur), the whole range of sea states — from very light wave conditions to severe storms — belongs essentially to one and the same weather population. Consequently, the short-term distribution of GSRs for each sea state and the long-term distribution of GSRs, being the weighted summation of all the short-term distributions, are also of this type. Generally, the distribution is well approximated by a Weibull distribution.

In tropical areas, both wave conditions due to normal atmospheric conditions (including winter storms and monsoon conditions) and wave conditions due to tropical cyclones (depending on the area, *cyclones* are called *hurricanes* or *typhoons*) belong to two different populations. This results in two different types of short-term distributions of GSRs for sea states belonging to the two populations. In these circumstances, the long-term distribution of local GSRs can be approximated by the combination of two Weibull distributions.

The applicability of these simplified methods is limited; see 16.9 for further information.

16.7 Determining the long-term stress range distribution by spectral analysis

16.7.1 General

Spectral analysis is a practical method that is best able to represent the random nature of the wave environment. Accordingly, the spectral method is the most comprehensive and most reliable assessment method for fatigue due to wave action. As it is based on superimposition of many individual frequency components, it is, strictly speaking, only applicable to linear systems. However, by suitable linearization of non-linear elements in the problem formulation (principally the drag term in Morison's equation and wave inundation effects around the still water level), this formal constraint can be adequately overcome.

The spectral method is performed in the frequency domain and explicitly accounts for the influence of the frequency of excitation (the wave frequency) on actions as well as on structural response.

The two crucial steps in the procedure are the determination of the frequency response functions (also referred to as transfer functions) for all GSRs and the subsequent calculation of the short-term response statistics in each sea state. These steps are described in 16.7.2 and 16.7.3, respectively, together with the information provided in A.16.7.

The final step in the procedure is to determine the long-term response distribution by accumulating the short-term response statistics for all sea states in the scatter diagram; see 16.7.4.

16.7.2 Stress transfer functions

16.7.2.1 General

When a linear system is subjected to harmonic excitation at a particular frequency (the input), the response of the system (the output) is also harmonic with the same frequency and a phase shift between input and output. The transfer function is defined as the ratio of the amplitude of the output to the amplitude of the input. This ratio is also known as the response amplitude operator (RAO). The complete operator between the input and the output, comprising both the RAO and the phase shift, is referred to as the frequency response function. Both the RAO and the phase shift are frequency dependent. The frequency response function is thus a complex valued function of frequency. The transfer function (or RAO) is the modulus of the frequency response function and hence a real function of frequency. Both the frequency response function and the transfer function are system properties and do not depend on the magnitude of the excitation. The distinction in terminology is, however, not always adhered to and the terms *frequency response function* and *transfer function* are often considered as synonyms. This practice will be followed in this International Standard as well: the transfer function can hence consist of the RAO and phase shift or of the RAO alone, depending on circumstances.

During the calculations, phase shifts are very important and shall be strictly maintained in order to obtain the correct response values. In the global analyses for a fatigue assessment of the in-place situation, the single input is the water surface elevation (the wave) and the multiple outputs are the nominal stresses at all locations of interest in the structure. All calculated nominal stresses are functions of wave frequency as well as wave approach direction.

The stress transfer functions may be determined by performing global stress analyses directly in the frequency domain for the real (in-phase) and imaginary (out-of-phase) parts of the applied wave action separately. If this method is chosen, the global analyses shall be performed using linear wave theory and the drag term in Morison's equation shall be used in a linearized form. The calculated stresses are then linearly dependent on the wave amplitude (height) and non-linear wave height influences are not included. Guidance on the selection of wave frequencies is given in 16.7.2.2.

Alternatively, the stress transfer functions may be determined by performing global stress analyses in the time domain by stepping a full wave cycle past the structure. If this method is chosen, the global analyses may be performed using various wave theories. The use of linear (Airy) wave theory is in strict accordance with the linearity of spectral analysis. A sinusoidal wave input to the structural analysis produces sinusoidal stress outputs. However, for the determination of transfer functions in engineering applications, the linearity demand can be somewhat relaxed. Use of non-linear waves (e.g. stretched Airy waves, Stokes V waves) and a non-linear drag term can be allowed. The wave input to the structural analysis is then no longer sinusoidal, and/or the model for the hydrodynamic actions is no longer fully linear, as a result of which the stress outputs are no longer purely sinusoidal either.

Using this relaxation it is possible to approximate the influence of non-linear wave actions on the calculated stresses by an appropriate selection of the wave height input and suitable linearization of the results. Guidance on the selection of wave heights to achieve this is given in 16.7.2.3. The selection of wave frequencies is the same as for the calculations in the frequency domain and is described in 16.7.2.2.

Once the transfer functions of the GSRs (see 16.5) have been determined, the RAO is the only variable that is needed in the further calculations.

16.7.2.2 Selection of wave frequencies

The frequencies of the regular waves used in the determination of the transfer functions shall be selected so that the transfer functions adequately define response features due to applied wave excitation as well as structural behaviour over the entire frequency range of interest. The selection shall hence be based on the characteristics of the environment as well as on the characteristics of the structure. Guidance on the selection is given in A.16.7.2.2.

The outcome of the further numerical calculations depends critically on the definition of the transfer functions. Therefore, their adequacy shall be checked. This can be done by plotting the transfer function of total applied actions on the structure (statically and dynamically) that are due to waves for each wave direction to ensure good definition of peaks and valleys and to verify the reliability of interpolation routines, if these are used. A Figure in A.16.7.2.2 shows an example of the shape of transfer functions of total applied actions and depicts typical features of the frequency grid selection.

16.7.2.3 Selection of wave heights

Where applicable, the heights of the regular waves used in the determination of the transfer functions shall be selected in a suitable manner, so that an appropriate level of non-linear (drag) action due to waves is included in the linearized calculations. This is normally achieved by means of choosing a constant wave steepness, which provides a simple relationship between the wave height and the wave frequency. Typical wave steepness values are in the range of 1:15 to 1:25. The value used can, for example, be determined by calibration for the structure under consideration in the applicable environment. For this calibration, the sea state at the centre of the fatigue damage scatter diagram may be used, which is representative of the conditions that contribute most to fatigue damage; see A.16.7.2.3.

NOTE Wave steepness is the ratio of wave height to wave length; the wave length is related to the wave frequency by the appropriate wave theory that is used in the analysis.

16.7.3 Short-term stress range statistics

Once the GSR transfer functions for one wave direction and all locations in the structure have been determined, short-term GSR statistics shall be calculated for each sea state using standard statistical procedures for Gaussian processes. The short-term statistics give the number of times that the GSR exceeds a certain value during one occurrence of the sea state and the wave approach direction considered. This shall be repeated for all sea states in the wave scatter diagram and for all wave approach directions.

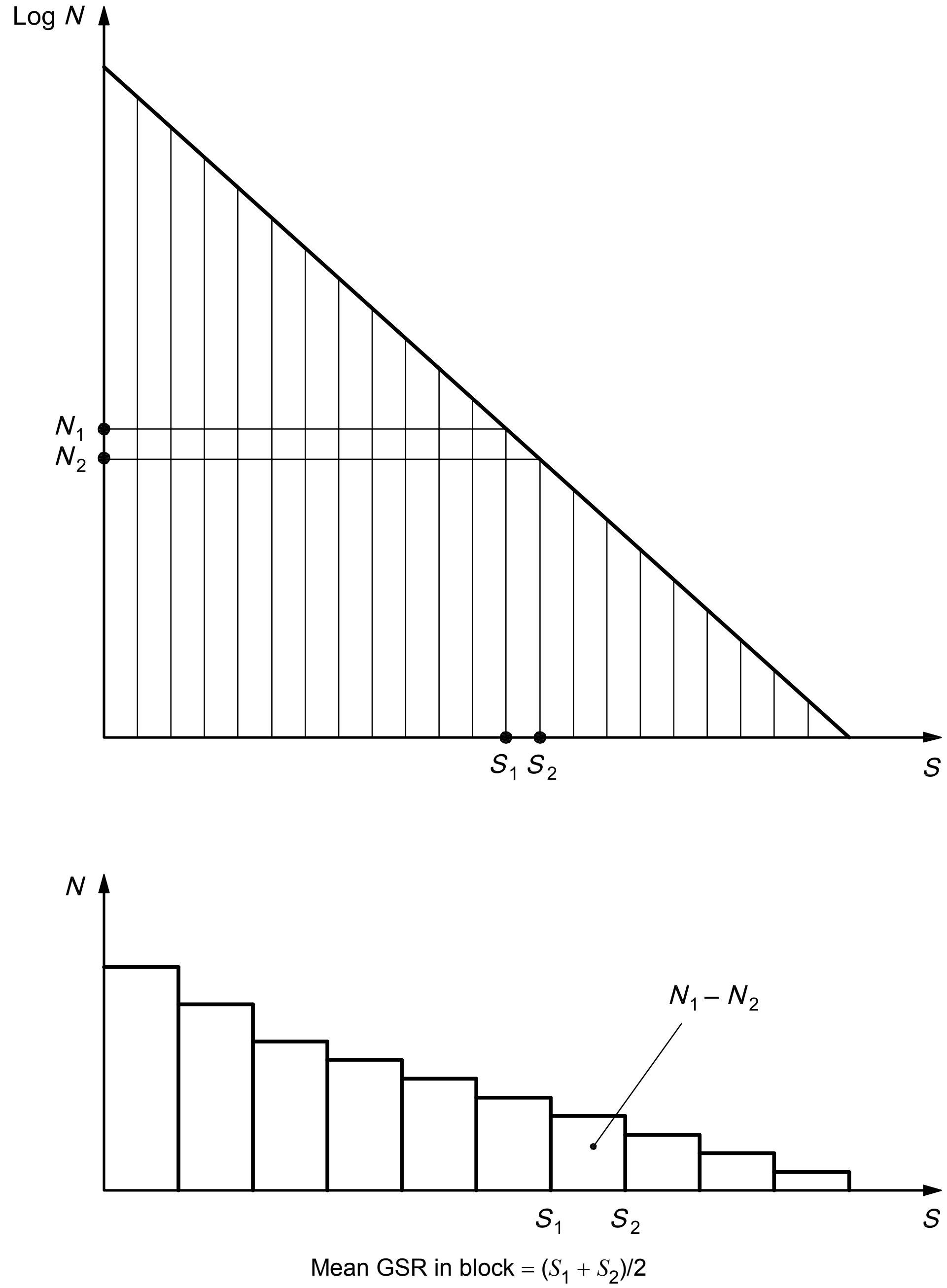
16.7.4 Long-term stress range statistics

The long-term GSR statistics for each location in the structure shall be calculated by accumulating the corresponding short-term GSR statistics over all sea states in the wave scatter diagram and all wave approach directions. Prior to summation, the short-term GSR statistics for an individual sea state and direction are weighted by the product of the probability of occurrence of the sea state and the probability of occurrence of the wave approach direction.

From the long-term cumulative distribution of GSR values, the numbers of occurrences in discrete GSR blocks are determined, which are used in the fatigue damage calculation as described in 16.12. An example of a long-term cumulative distribution of GSR values and its subdivision into discrete GSR blocks is shown in Figure 16.7-1. The distribution is usually found to be close to an exponential distribution and plots approximately as a straight line on log-linear scales.

NOTE 1 A legitimate alternative procedure that is sometimes followed is to calculate the fatigue damage per sea state directly from the short-term GSR statistics. If this is done, these partial fatigue damages are summed after weighting them by the product of the probability of occurrence of the sea state and the probability of occurrence of the wave approach direction.

NOTE 2 In some computer programs, the fatigue damage per sea state is calculated using a closed-form solution based on analytical expressions for the short-term GSR statistics and the applicable *S–N* curve. Provided the closed-form procedure can handle bilinear *S–N* curves, where applicable, this is another legitimate procedure.



Key

S stress range at location, $S = C \times S_{nom} = C \times (\sigma_{max} - \sigma_{min})$, linear scale

where

C is the stress concentration factor;

σ_{max} and σ_{min} are the maximum and minimum nominal stresses.

N number of cycles, logarithmic scale

$N_1 - N_2$ number of cycles in GSR block being considered

N_1 number of cycles by which stress range exceeds S_1

N_2 number of cycles by which stress range exceeds S_2

S_1 lower value of stress range in GSR block being considered

S_2 upper value of stress range in GSR block being considered

Figure 16.7-1 — Typical long-term cumulative distribution of GSR values

16.8 Determining the long-term stress range distribution by deterministic analysis

16.8.1 General

The deterministic fatigue analysis procedure uses an artificial representation of the true random nature of the wave environment. The method has similarities with conventional design wave analyses in that environmental actions and structural responses are calculated directly by periodic (regular) waves. The method can offer some computational saving compared with a spectral analysis, but it still requires a substantial computational effort.

A deterministic fatigue analysis method is not recommended for final checking of structures in harsh fatigue environments. It can find application for screening evaluations during the initial design stages, or for a final fatigue assessment of structures for which dynamic effects can be neglected and that are not critically fatigue sensitive. It is included in this International Standard only in order to cover such general applications. Guidance on dynamically responding structures is given in 16.4.3 and 16.6.4.

However, by exception, members in the splash zone between the highest submerged plan level and the lowest plan level above still water are often best analysed by means of a deterministic fatigue analysis; see also 16.14.6.

The deterministic fatigue analysis method uses the long-term distribution of individual wave heights to represent the long-term wave environment; see 16.3.7. Stresses are calculated in a series of analyses, using deterministic waves with wave heights that are selected from this distribution (see 16.8.2) and wave periods that are appropriate for the chosen wave height classes (see 16.8.3).

The influence of non-linear effects in the applied wave actions for the selected regular waves, as well as non-linear aspects of structural behaviour, can be included in the analysis.

16.8.2 Wave height selection

The periodic (regular) waves used for the deterministic analysis are selected to represent the long-term distribution of individual wave heights. This distribution is discretized into a number of wave height blocks. Typically, a minimum of 6 to 10 blocks with uniform increments in wave height are selected. Assuming that omni-directional wave height statistics are used, then the same set of wave heights will apply for all wave directions.

16.8.3 Wave period selection

The wave periods for the selected regular wave heights shall be determined, based on the expected joint distribution of wave heights and periods for the location. Considerations similar to the selection of wave frequencies for the spectral method apply (see 16.7.2.2).

16.8.4 Long-term stress range distribution

For each location in the structure, the long-term distribution of GSR values is calculated by accumulating the GSR values from all regular wave analyses. Assuming that omni-directional wave height statistics are used, the total number of GSR occurrences for each wave height block is the sum of the number of occurrences in the wave height block, weighted with the probability of occurrence of the wave approach direction.

16.9 Determining the long-term stress range distribution by approximate methods

As noted in 16.6.4, the long-term distribution of local geometric stress ranges (GSRs) can in many cases be represented by a Weibull distribution. The Weibull distribution is a very general class of statistical distributions by which natural phenomena (e.g. wave heights, wind speeds) and engineering applications (e.g. reliability problems) can be suitably described. See A.16.9 for a discussion of the Weibull distribution, the determination of its parameters and their application to fatigue assessments.

Methods using estimated Weibull distributions are intended for initial screening of structures, or for providing supposedly conservative fatigue checks of structures in benign fatigue environments. The methods are not suitable for final checking of structures in harsh fatigue environments or for structures that respond dynamically; see also 16.2.7.

The methods can also be used to establish approximate allowable GSR values, which may be used as a guide for initial joint design. The expected adequacy of the structure during the design process in satisfying final fatigue requirements is then assessed by comparing an allowable design value of the local GSR with the GSR calculated by stepping the associated fatigue design wave through the structure (see A.16.9). The final design should be checked by performing a spectral or a deterministic fatigue analysis, as appropriate.

Due to the increase in computing power and the availability of computer programs for performing detailed fatigue analyses in a relatively straightforward and rapid manner the relevance of, and interest in, approximate procedures for determining the long-term stress range distribution has distinctly diminished.

16.10 Geometrical stress ranges

16.10.1 General

The stress parameter that shall be used to evaluate fatigue damage at any weld is the geometric stress range (GSR), see 16.5.

16.10.2 Stress concentration factors for tubular joints

16.10.2.1 General requirements for the determination of the stress concentration factor

The welds at tubular joints are among the most fatigue-sensitive areas in offshore structures, due to the high local stress concentrations. Fatigue damage at locations around the connection weld shall be estimated by evaluating the GSR using the associated SCFs at each location of interest. Specific information and guidance is provided in A.16.10.2.1.

The nominal brace stress shall be based on the section properties of the brace end under consideration, taking due account of the brace stub or a flared member end, if present. The evaluation of the SCF shall be based on the same section dimensions. Nominal variable stress in the chord can also influence the GSR and shall be considered; see A.16.10.2.1.5.

The SCF shall include all stress raising effects associated with the geometry and force pattern of the joint, except the local (microscopic) weld notch effect, which is included in the S–N curve.

SCFs may be derived from FEA, model tests or empirical equations based on such methods. In general, the SCFs depend on the type of variable forces exerted by the brace on the joint, the type of joint, and details of the joint geometry; see the classification of tubular joints into types in Clause 14 and in A.16.10.2.1.3. Brace axial forces, brace in-plane bending (ipb) and brace out-of-plane bending (opb) moments cause different patterns of variable stresses around the joint.

For welded tubular joints subjected to all three types of variable forces and moments, a minimum SCF of 1,5 shall be used. For ring-stiffened joints, the minimum SCF for the brace side of the weld and brace axial forces or brace opb moments shall be taken as 2,0.

16.10.2.2 Unstiffened tubular joints

General guidance and discussion of SCFs for unstiffened welded tubular joints is given in A.16.10.2.2. The SCFs may be evaluated using the Efthymiou equations, see A.16.10.2.2.2. For this purpose, the tubular joints are typically classified into T/Y-, X-, K- and KT-joints, depending on the joint configuration, the brace under consideration and the brace force pattern, as described in A.16.10.2.1.

As a generalization of the classification approach, the influence function algorithm discussed in A.16.10.2.2.4 may be used to evaluate the GSRs. This algorithm can handle generalized forces and moments on the braces. Moreover, the influence function algorithm can handle multiplanar joints for the important case of axial forces.

For a discussion on tubular joints welded from one side, see A.16.10.2.2.5.

Thickness transitions between butt welded tubular members are also subject to stress concentrations. This can apply to all joint-can-to-chord and brace-stub-to-brace connections. It is recommended that thickness transitions be designed to be internally flush; see A.16.10.2.2.6. The transition should normally have a slope of 1:4. Similarly, diameter transitions achieved through the use of a truncated cone are subject to stress concentrations; see 13.6.

16.10.2.3 Internally ring stiffened tubular joints

The SCF concept also applies to internally ring-stiffened joints, including the stresses in the stiffeners and the stiffener to chord weld. Ring-stiffened joints can have stress peaks at the brace to ring intersection points. Special consideration should be given to these locations. SCFs for internally ring-stiffened joints can be determined by applying the Lloyds reduction factors to the SCFs for the equivalent unstiffened joint, see A.16.10.2.3. Ring stiffeners without flanges on the internal rings should be used with care, since high stress concentrations can occur at the inner edge of the ring.

16.10.2.4 Grouted tubular joints

Grouting tends to reduce the SCF of the joint since the grout reduces the chord deformations. In general, the larger the ungrouted SCF, the greater the reduction in SCF for grouted joints. Hence, the reductions are typically higher for X- and T-joints than for Y- and K-joints. Approaches for calculating SCFs for grouted joints are discussed in A.16.10.2.4.

16.10.2.5 Cast joints

For cast joints, the SCF is defined as the maximum principal stress at any point on the surface of the casting (including the inside) divided by the nominal brace stress outside the casting. The SCFs for castings are not extrapolated values, but are based on directly measured or calculated values at any given point. Consideration shall be given to the brace to casting circumferential butt weld, which can be the most critical location for fatigue.

16.10.3 Geometric stress ranges for other fatigue sensitive locations

This subclause covers all types of fatigue-sensitive locations other than tubular joints, which are considered in 16.10.2. These locations include but are not limited to material away from the weld, plate to tubular connections and weld attachments. Each location at which a fatigue crack can potentially develop shall be classified according to the appropriate type of constructional detail, see A.16.10.3.

For non-tubular connections, the GSR is obtained by assigning a joint classification to the plain metal, to the connection (or to a constructional detail of a connection) and specifying which stress range in the vicinity of the detail is considered the most appropriate for the local geometry of the class and the type of variable forces involved. This stress range shall be used as the GSR. Any additional stress-raising effect due to the gross geometry of the joint (e.g. stress concentrations resulting from holes, shear lag or local through-wall bending) should be separately included where appropriate.

A procedure for the fatigue assessment of bolts is given in Clause 15.

16.11 Fatigue resistance of the material

16.11.1 Basic S–N curves

The basic representative S–N curve is of the form:

$$\log_{10}N = \log_{10}k_1 - m \log_{10}S \qquad (16.11\text{-}1)$$

where

N is the predicted number of cycles to failure under constant amplitude stress range, S;

k_1 is a constant ($k_1 = N$ for $S = 1$);

m is the inverse slope of the S–N curve;

S is the constant amplitude stress range, which is the geometrical stress range according to 16.10.

Representative S–N curves for tubular joints (TJ), cast joints (CJ) and other joints (OJ) are given in Table 16.11-1. These S–N curves are based on steels with a yield strength less than 500 MPa; see 16.11.2 for steels with higher yield strengths.

The representative S–N curves presented in Table 16.11-1 are the mean minus two standard deviations in $\log_{10}N$ from the database of test results for each class. The curves are applicable for joints in both air and sea water with adequate corrosion protection. Information concerning S–N curves for joints without adequate protection is given in A.16.11.1.

For welded tubular joints exposed to constant amplitude or random variations of stress due to environmental or operational actions, the TJ curve shall be used. The brace to chord tubular intersection for ring-stiffened joints shall also be designed using the TJ curve. Fabrication of welded joints shall be in accordance with Clause 20.

The set of OJ S–N curves shall be used for joints classified in 16.10.3, including plated joints. For ring-stiffened joints, the ring stiffener to chord connection as well as the ring inner edge shall also be designed using the OJ curve.

Fatigue crack growth from the weld root into the section under the weld is possible and should be considered. This type of cracking is referred to in A.16.10.3, with examples in Table A.16.10-8.

For cast joints see 16.11.3.

Table 16.11-1 — Basic representative S–N curves for air and sea water
(with adequate corrosion protection)

Curve	Air		Sea water with adequate corrosion protection	
	$\log_{10}k_1$ (for S in units of MPa)	m	$\log_{10}k_1$ (for S in units of MPa)	m
Tubular joints (TJ)	12,48 16,13	3,0 for $N \leqslant 10^7$ 5,0 for $N > 10^7$	12,18 16,13	3,0 for $N \leqslant 1,8 \times 10^6$ 5,0 for $N > 1,8 \times 10^6$
Cast joints (CJ)	15,17	4,0	See A.16.10.2.5 and A.16.11.3	
Other joints (OJ)				
B	15,01 17,01	4,0 for $N \leqslant 10^7$ 5,0 for $N > 10^7$	14,61 17,01	4,0 for $N \leqslant 10^5$ 5,0 for $N > 10^5$
C	13,63 16,47	3,5 for $N \leqslant 10^7$ 5,0 for $N > 10^7$	13,23 16,47	3,5 for $N \leqslant 4,68 \times 10^5$ 5,0 for $N > 4,68 \times 10^5$
D	12,18 15,63	3,0 for $N \leqslant 10^7$ 5,0 for $N > 10^7$	11,78 15,63	3,0 for $N \leqslant 10^6$ 5,0 for $N > 10^6$
E	12,02 15,37	3,0 for $N \leqslant 10^7$ 5,0 for $N > 10^7$	11,62 15,37	3,0 for $N \leqslant 10^6$ 5,0 for $N > 10^6$
F	11,80 15,00	3,0 for $N \leqslant 10^7$ 5,0 for $N > 10^7$	11,40 15,00	3,0 for $N \leqslant 10^6$ 5,0 for $N > 10^6$
F_2	11,63 14,71	3,0 for $N \leqslant 10^7$ 5,0 for $N > 10^7$	11,23 14,71	3,0 for $N \leqslant 10^6$ 5,0 for $N > 10^6$
G	11,40 14,33	3,0 for $N \leqslant 10^7$ 5,0 for $N > 10^7$	11,00 14,33	3,0 for $N \leqslant 10^6$ 5,0 for $N > 10^6$
W_1	10,97 13,62	3,0 for $N \leqslant 10^7$ 5,0 for $N > 10^7$	10,57 13,62	3,0 for $N \leqslant 10^6$ 5,0 for $N > 10^6$
See A.16.10.3 and Tables A.16.10-6 to A.16.10-11 for application of the relevant OJ curve from B to W_1 to a particular constructional detail.				

16.11.2 High strength steels

There is generally insufficient data on the fatigue behaviour of high strength steels to be able to confidently predict the fatigue performance of higher strength steels; see A.16.11.2 for some information.

16.11.3 Cast joints

For cast joints, the CJ curve in Table 16.11-1 shall be used. The curve for cast joints is only applicable to castings having an adequate fabrication inspection plan.

16.11.4 Thickness effect

The TJ and OJ curves presented in Table 16.11-1 are based on 16 mm material thickness. For material thickness above 16 mm, the following thickness effect shall be applied:

$$S = S_0(16/t)^{0,25} \tag{16.11-2}$$

where

S is the stress range of the S–N curve, when adjusted for thickness effects;

S_0 is the stress range of the S–N curve in Table 16.11-1;

t is the member thickness in mm for which the fatigue life is predicted.

The CJ curve is based on 38 mm material thickness. For material thickness above 38 mm, the following thickness effect shall be applied:

$$S = S_0(38/t)^{0,15} \qquad (16.11\text{-}3)$$

No thickness effect shall be applied to material thicknesses less than 16 mm for the TJ and OJ curves and less than 38 mm for the CJ curve.

16.12 Fatigue assessment

16.12.1 Cumulative damage and fatigue life

Fatigue assessment due to variable actions is normally based on the hypothesis of a linear accumulation of fatigue damage under constant amplitude stress ranges, according to the Palmgren-Miner rule:

$$D = k_{\mathrm{LE}} \cdot \gamma_{\mathrm{FD}} \cdot \sum_{\mathrm{i}} \frac{n_{\mathrm{i}}}{N_{\mathrm{i}}} \qquad (16.12\text{-}1)$$

where

D is a non-dimensional number, the Palmgren-Miner sum or damage ratio for a time T;

k_{LE} is a local experience factor, see 16.12.3;

γ_{FD} is a fatigue damage design factor, see 16.12.2;

n_{i} is the number of cycles of stress range, S_{i}, occurring during time period, T;

N_{i} is the number of cycles to failure under constant amplitude stress range, S_{i}, taken from the relevant S–N curve.

In the fatigue assessment procedure, fatigue failure is assumed to occur when $D = 1,0$.

Equation (16.12-1) may be employed for

a) variable stress ranges associated with a given sea state,

b) multiple sea states and wave directions, and

c) multiple sources of cyclic actions.

It may also be used to evaluate multiple sets of *in situ* conditions, such as for a structure that has been relocated.

Equation (16.12-1) may be used to estimate fatigue life. If all damage is due to the same circumstances (e.g. in the in-place situation), the calculated fatigue life L may be calculated using Equation (16.12-2):

$$L = T/D \qquad (16.12\text{-}2)$$

where

L is the calculated fatigue life based on the calculated fatigue damage;

T is the time period ($T \geqslant 1$ year) over which the Palmgren-Miner sum was determined;

D is the calculated fatigue damage during time, T, according to the Palmgren-Miner sum.

If some prior fatigue damage, D_1, has occurred, Equation (16.12-2) shall be applied to the remaining fatigue damage, D_2. As failure is assumed to occur when $D_1 + D_2 = 1{,}0$, the allowable remaining fatigue damage is $D_2 = (1{,}0 - D_1)$. If D_2 is associated with a time period, T_2, over which the Palmgren-Miner sum is determined, the remaining fatigue life, L_2, is given by Equation (16.2-3):

$$L_2 = T_2/(1{,}0 - D_1) \qquad (16.12\text{-}3)$$

The prior damage, D_1, can, for example, be due to damage during fabrication in the construction yard, or damage during the tow to location, or damage due to prior structure use.

16.12.2 Fatigue damage design factors

For *in situ* conditions, the fatigue damage design factors, γ_{FD}, for fatigue of steel components primarily depend on the failure consequence (i.e. the component's criticality) and in-service inspectability. In lieu of more detailed assessment, a γ_{FD} of 2,0 is recommended for inspectable and non-failure critical locations. For failure-critical and/or non-inspectable locations, γ_{FD} values of up to 10 are recommended, see A.16.12.2. *Inspectability* implies meeting the appropriate in-service inspection requirements as detailed in Clause 23. For new designs, the γ_{FD} shall not be taken as less than 1,0.

16.12.3 Local experience factor

As stated in several places, fatigue analyses and assessment procedures are subject to many and considerable uncertainties. Therefore, actual experience with existing structures is of great value and may justifiably be taken into account through the application of an overall local experience factor, k_{LE}, provided this experience is directly relevant to the case being considered, can be substantiated by reliable evidence and is fully documented; see A.16.12.3.

16.13 Other causes of fatigue damage than wave action

16.13.1 General

Wave action in the in-place situation is the predominant source of fatigue damage to offshore structures. However, other sources of fatigue damage should also be given consideration. The most relevant ones are described in this subclause. If different damage sources are not acting concurrently, damage due to each source can in general be evaluated separately and damage ratios can be added.

16.13.2 Vortex induced vibrations

Structural members that are susceptible to vortex induced vibrations (VIV) due to either wind, current or waves can experience large cyclic stresses causing fatigue damage. The susceptibility of components to VIV shall be assessed. For those members for which it cannot be confidently ensured that they will not experience VIV, an evaluation of vibration amplitudes, stresses and associated fatigue damage shall be undertaken.

16.13.3 Wind induced vibrations

During construction in the fabrication yard some members can experience vibrations due to wind. If this occurs, the associated fatigue damage shall be assessed.

Flexible topsides structures such as vent or flare structures with comparatively long natural periods can be susceptible to resonant vibrations due to wind gust effects. The resulting stresses can be evaluated using dynamic spectral analysis techniques using a suitable wind spectrum. If these stresses are significant the resulting fatigue damage shall be evaluated.

16.13.4 Transportation

Structures with long, exposed ocean tows from the fabrication yard to the in-place location shall be checked for fatigue damage arising during transportation. The fatigue damage during tow in all components of the structure shall be combined with the fatigue damage from other sources using suitable fatigue damage design and/or experience factors to account for uncertainties in its determination. Sea-fastening members shall be included in the fatigue damage assessment.

16.13.5 Installation

The main source of cyclic stresses during installation is due to pile driving. If hard driving is expected, the fatigue damage in piles and conductors due to driving shall be evaluated and combined with the fatigue damage from other sources. Information on the expected number of hammer blows and the magnitude and number of stress cycles per blow can be determined from the driveability analysis and shall be used as the basis for the fatigue calculation. For pile driving analyses, worst case soil conditions should be applied.

During pile driving, large cyclic stresses can further occur due to vibrations of miscellaneous attachments such as anodes and installation piping in the vicinity of the pile, pile sleeves and hammer. Rigorous analysis of these vibrations is usually not feasible, but simplified assessments using appropriate assumptions on peak accelerations, dynamic amplification and number of vibration cycles can provide insight into the severity of the problem. In practice, design measures to avoid the potential fatigue problem are most effective.

16.13.6 Risers

The design of risers normally requires input from both the structure designer and the pipeline designer. Risers will normally be modelled in the structure analysis to assess fatigue damage due to wave action on the risers; see also 16.14.2. This damage shall be combined with fatigue damage due to other sources that are normally assessed by the pipeline designer, including damage due to pipeline start-up and shut-down forces and slug induced forces. Variations in the actions on the riser often also act on the riser supports and these shall hence be included in the assessment.

16.14 Further design considerations

16.14.1 General

In addition to the description of the procedures to determine the fatigue resistance during design (see 16.11 and 16.12), close attention to good detailing for both the main structure and all significant constructional details is of great importance in ensuring that adequate fatigue performance is achieved.

In 16.14, some aspects requiring particular attention during design and analysis with regard to fatigue are discussed.

Furthermore, the approach normally followed in fatigue evaluations is inherently linked to fabrication and inspection methods and to the in-service inspection philosophy. Some guidance in relation to these aspects is given.

16.14.2 Conductors, caissons and risers

To check the fatigue of conductors, caissons and risers and their supports, it is normally necessary to include a detailed structural model of these components in the overall structural analysis. Where there are several components with a standard arrangement it can be adequate to model only one typical example of each in detail. Depending on structural connection details, these components can also contribute stiffness to the members of the space frame and thereby influence the stresses, and hence the fatigue damage experienced, by the members in the frame(s) to which they are connected.

16.14.3 Miscellaneous non-load carrying attachments

To check the structure for miscellaneous non-load carrying attachments, such as brackets or padeyes which are left from construction, all members of the structure shall be screened using the F_2 S–N curve in conjunction with a GSR that is equal to the maximum combination of nominal stresses in the member (i.e. all SCFs equal to unity).

16.14.4 Miscellaneous load carrying attachments

It is recommended that connections of miscellaneous attachments such as anodes, ladders, walkways, minor piping, etc., which are subjected to local wave action and are not explicitly modelled in the structural analysis, are separately checked for possible fatigue damage. Simplified evaluations based on the principles of deterministic analyses are normally adequate for these applications, using realistic numbers of cycles and action magnitudes.

Where the connections of attachments to primary members are found to be sensitive to fatigue, doubler plates are recommended. Doubler plates reduce the stresses in the primary member, and any cracking due to the attachment to the doubler plate is remote from the primary member.

16.14.5 Conical transitions

See 13.6 for stresses in conical transitions.

16.14.6 Members in the splash zone

Fatigue damage for members close to the water surface is subject to additional uncertainty due to the complexities of local actions (see A.16.14.6) and sensitivity of the actions to the actual structure elevation, both initially and because this can change during the design service life. Where uncertainty exists in the water depth or final structure settlement, all members in the splash zone, the actions on which can be significantly affected by this uncertainty, shall be designed with an increased fatigue damage design factor or by explicitly assessing the possible range of water depths.

16.14.7 Topsides structure

Topsides structures supported by conventional steel jacket or tower structures do not normally experience cyclic stresses from wave action. However, exceptions can occur. For example, for structures with an unbraced portal arrangement supporting the topsides structure, cyclic wave action on the structure can introduce significant bending moments and associated variable stresses into the topsides structure. Another exception concerns conductors or other components that span from the water surface to the deck without intermediate supports.

Where the configuration is such that the deck can be subjected to the effects of cyclic wave action, fatigue of the topsides structure shall be evaluated. This evaluation shall take appropriate account of the effect of wave frequency on wave actions applied to structural components in the wave zone. This normally requires that a spectral analysis is performed.

For structures that are subject to significant dynamic response, the mass inertia forces of the topsides can cause significant cyclic stresses in the topsides structure. This can, for example, be the case for structures with relatively long natural periods and associated large dynamic excitation. In such cases, fatigue of the topsides structure shall be considered. This will normally require that a spectral analysis be performed of the combined structure and topsides structure.

16.14.8 Inspection strategy

The predicted fatigue performance of structural components provides valuable input for establishing a strategy for underwater inspection, in addition to other considerations that need to be considered when formulating such a strategy. However, designing new structures with a comfortable margin in calculated fatigue lives generally provides for a cost effective investment as compared with a marginal design requiring more extensive inspection for fatigue.

16.15 Fracture mechanics methods

16.15.1 General

Fracture mechanics methods may be employed to quantify fatigue lives of welded details or structural components in situations where the normal S–N fatigue assessment procedures are inappropriate. Some typical applications are

- to assess the fitness-for-purpose of a component with or without known defects,
- to assess the inspection requirements for a component with or without known defects,
- to assess the inspection requirements for components which may not be subjected to post-weld heat treatment (PWHT), and
- to assess the structural integrity of castings.

It is important that the fracture mechanics formulation used can be shown to predict, with acceptable accuracy, either

- the fatigue performance of a component class with a detail similar to that under consideration, or
- test data for components which are similar to those requiring assessment.

16.15.2 Fracture assessment

The principal modes of failure in offshore structures are

- crack growth driven by fatigue followed by the onset of fracture due to exceedance of the fracture toughness at a critical crack size (not necessarily through-thickness), and
- the occurrence of plastic collapse.

16.15.3 Fatigue crack growth law

Using fracture mechanics, the predicted number of cycles to failure (providing the fatigue life) is determined by the integration of a suitable fatigue crack growth law, which relates the fatigue crack growth rate to the stress intensity factor range. The relationship is usually of a sigmoidal shape with a linear central region which is defined by the Paris law:

$$\mathrm{d}a/\mathrm{d}N = C\,(\Delta K)^m \qquad (16.15\text{-}1)$$

where

a is the crack depth;

N is the number of cycles to failure;

ΔK is the stress intensity factor range;

C and m are parameters of the crack growth rate.

The stress intensity factor range is given by the general expression:

$$\Delta K = Y\left(\Delta\sigma\right)\sqrt{\pi a} \qquad (16.15\text{-}2)$$

where, additionally,

Y is the normalized stress intensity factor;

$\Delta\sigma$ is the stress range.

It is normal practice to extrapolate the Paris law to all ranges of ΔK. The number of cycles to failure is thus determined by integrating Equation (16.15-1):

$$N = \int_{a_i}^{a_f} \frac{\mathrm{d}a}{C\left[Y\left(\Delta\sigma\right)\sqrt{\pi a}\right]^m} \tag{16.15-3}$$

where, further,

a_i is the initial crack size;

a_f is the final crack size.

Guidance on the integration is provided in A.16.15.3.

16.15.4 Stress intensity factors

Stress intensity factors for tubular joints can be determined by a number of different methods; see A.16.15.3.

16.15.5 Fatigue stress ranges

For a crack growth analysis using fracture mechanics methods, the stress ranges at the location of the crack tip are required, which differ from the GSR used for an S–N assessment. Both are derived from the same nominal axial and bending stresses obtained from the global structural analyses. However, while the GSR is determined by applying appropriate SCFs to the nominal stress components (see 16.10), the stress field at the crack tip is determined by applying Equation (16.15-2) with the appropriate stress intensity factors to the nominal axial and bending stresses. As for the GSR, the full stress range at the location of the crack is next calculated by combining the local axial and bending stresses.

16.15.6 Castings

An advantage of using castings is that stress concentrations can be minimized by profiling the corners between parts of the casting. Consequently, higher fatigue lives than for similar fabricated joints are obtained. However, castings are susceptible to internal and surface casting defects. Fitness-for-purpose assessments of castings shall therefore include a fracture mechanics assessment.

16.16 Fatigue performance improvement of existing components

For welded joints, improvements of fatigue performance can be obtained by a number of methods; these include hammer peening and controlled local machining or grinding of the weld toe to produce a smooth concave profile, which blends smoothly with the parent metal. Table 16.16-1 indicates associated improvement factors on fatigue life that can be achieved. Adequate quality control procedures shall be applied if the appropriate improvement factor is to be attained.

The grinding improvement factor is not applicable to joints in sea water without adequate cathodic protection.

Benefit shall only be taken for one improvement technique, even if more than one technique is employed.

Specific requirements for the various weld improvement techniques are discussed in A.16.16.

Table 16.16-1 — Achievable improvement factors on fatigue performance for weld improvement techniques

Weld improvement technique	Improvement factor
Weld toe burr grinding	2
Hammer peening	4

17 Foundation design

17.1 General

17.1.1 Applicability

This clause establishes requirements for foundation design. Pile foundations and, more specifically, steel cylindrical (pipe) pile foundations are addressed in 17.1 to 17.11. Considerations for the design of shallow foundations are given in 17.12, while design requirements and guidance can be found in ISO 19901-4. A.17 contains discussion and guidance on the requirements of Clause 17.

Additional guidance on matters relating to foundations is given in ISO 19900 and ISO 19901-4.

17.1.2 Overall considerations

In keeping with normal practice, simple design formulae are provided in this International Standard; these formulae have been widely used by the industry for routine and familiar situations. This does not mean that they are state-of-the-art or the best available, but they do have the advantage of using a few standard, easily derived soil parameters and of being the subject of extensive discussion in the technical literature. A literature review shows that in some circumstances they result in foundations with a calculated reliability lower than would normally be considered acceptable and, in other circumstances, in foundations which are overdesigned. Designers are encouraged to use relevant industry experience, some of which is mentioned herein, as well as their own experience, to determine whether the quoted formulae are appropriate for a given situation and to evaluate the suitability of alternatives. Alternative design procedures are permitted, always subject to such an evaluation. Circumstances can arise when neither the procedures herein, nor any existing alternative procedures satisfy the evaluation. The design shall then be conservative and flexible, and based on, and verified by, a combination of load testing, model tests and monitoring.

The foundation shall be designed to carry static, cyclic and transient actions without excessive deformations or vibrations in the structure. Special attention shall be given to the effects of cyclic and transient actions on the structural behaviour of piles, as well as on the strength of the supporting soils. The criteria for foundation capacity provided in 17.3, 17.4 and 17.5 are based upon static, monotonic actions. The foundation capacity refers to axial pile capacity only, as lateral soil behaviour is of sole interest for foundation displacements and not for capacity. Furthermore, these criteria do not necessarily apply to so-called “problem” soils, such as carbonate material, volcanic sands or highly sensitive clays. The possibility of movement of the sea floor against the foundation components shall be investigated. The actions caused by such movements, if anticipated, shall be considered in the design. The potential for disturbance to foundation soils by installation of well conductors or shallow well drilling shall also be assessed (see 22.6). The design of foundations differs from that of the rest of the structure in that the soil behaviour is non-linear as well as dependent on the rate of change of the actions.

The geotechnical investigation for pile-supported structures shall provide, as a minimum, the soil engineering property data needed to determine the following parameters: axial capacity of piles in tension and compression, force-deflection characteristics of piles subjected to axial and lateral actions, pile driveability characteristics, and mudmat bearing capacity. Mudmat design requires careful determination of the slope of the sea floor and accurate assessment of the shear strength of soft surface layers.

17.1.3 Exposure levels

It is not normal practice to apply the concept of exposure levels, as discussed in 6.6, to the foundation design for new structures. Therefore, the partial action factors that are applicable to exposure level L1 shall be used for the design of foundations for structures of all exposure levels. If enhanced or reduced reliability of the foundations is justified, a proper risk assessment shall be undertaken and the resistance factor varied accordingly. For existing structures, the use of reduced partial action factors for the assessment of exposure level L2 and L3 structures may be considered.

17.2 Pile foundations

17.2.1 Types of pile foundation

The types of pile foundation used to support offshore structures and considered in this International Standard are

a) driven piles,

b) drilled and grouted piles,

c) belled piles, and

d) vibro-driven piles.

17.2.2 Driven piles

Open-ended piles are normally used in foundations for offshore structures. These piles are usually driven into the seabed with impact hammers, which use steam, diesel fuel or hydraulic power as the source of energy. Driveability studies are carried out in accordance with the principles given in 22.5.5 in order to define the type of hammer necessary to reach the target design pile penetration.

The design penetration of driven piles shall be determined in accordance with the principles outlined in 17.3 to 17.6 and 17.9, rather than upon any correlation of pile capacity with the number of blows required to drive the pile a certain distance into the seabed.

17.2.3 Drilled and grouted piles

Drilled and grouted piles can be used in soils and rocks which will hold an open hole with or without drilling mud. The design penetration of drilled and grouted piles shall be determined in accordance with the principles outlined in 17.3 to 17.6. Force transfer between grout and pile shall be designed in accordance with 15.1, provided that the radial stiffness term for the pile sleeve is replaced by a stiffness term for the rock mass.

17.2.4 Belled piles

Bells may be constructed at the tip of piles so as to give increased bearing and uplift capacity through direct bearing on the soil. The end bearing capacity of belled piles shall be determined in accordance with the principles given for the design of drilled and grouted piles.

17.2.5 Vibro-driven piles

The capability of hydraulic vibratory driving hammers to install piles has been demonstrated, in particular for the installation of small diameter piles in cohesionless soils. Owing to the lack of data with respect to the effect of the installation method on the pile axial capacity, the use of vibratory hammers for installing piles subjected to significant axial actions is not recommended.

Vibratory hammers may be considered for installing well conductors, or piles which are predominantly subjected to horizontal actions, such as reaction piles for start-up of pipelines or anchor piles. They are also frequently used where extraction and repositioning can be required. Vibratory hammers may further be considered as complementary tools to impact hammers, i.e. for initial driving.

17.3 General requirements for pile design

17.3.1 Foundation size

When sizing a pile foundation, the following items shall be considered: design actions, diameter, penetration, wall thickness, type of tip, number of piles, spacing, geometry, location, pile head fixity, material strength, installation method, and other parameters as appropriate.

17.3.2 Foundation response

A number of different analysis procedures may be used to determine the requirements of a foundation. As a minimum, the procedure used shall properly simulate the non-linear behaviour of the soil and ensure force-deflection compatibility between the structure and the pile-soil system.

17.3.3 Deflections and rotations

Deflections and rotations of individual piles and the total foundation system shall be checked at all critical locations, which include pile tops, points of contraflecture, sea floor, etc. Deflections and rotations shall not exceed serviceability limits that if exceeded would render the structure inadequate for its intended function, see 7.2.

17.3.4 Foundation capacity

The foundation capacity is the lower of the pile strength and the pile axial resistance as below.

a) Pile strength

The pile strength shall be verified using the steel tubular strength checking equations given in 13.3 or 13.4 for conditions of combined axial force and bending. Internal pile forces at the location being checked shall be those due to the design actions according to 9.10 using a coupled structure/soil non-linear foundation model. When lateral restraint normally provided by the soil is inadequate or non-existent, column buckling effects on the pile shall also be checked in accordance with 17.10.2.

b) Pile axial resistance

The axial pile capacity shall satisfy the following conditions:

$$P_{d,e} \leqslant Q_d = Q_r/\gamma_{R,Pe} \tag{17.3-1}$$

$$P_{d,p} \leqslant Q_d = Q_r/\gamma_{R,Pp} \tag{17.3-2}$$

where

Q_d is the design axial pile capacity, i.e. the design resistance of the pile;

Q_r is the representative value of the axial pile capacity as determined in 17.4 and 17.5;

$P_{d,e}$ is the design axial action on the pile, determined from a coupled linear structure and non-linear foundation model using the design actions for extreme conditions according to 9.10.3;

$P_{d,p}$ is the design axial action on the pile, determined from a coupled linear structure and non-linear foundation model using the design actions for permanent and variable actions or the design axial action for operating situations according to 9.10.3;

$\gamma_{R,Pe}$ is the pile partial resistance factor for extreme conditions ($\gamma_{R,Pe}$ = 1,25);

$\gamma_{R,Pp}$ is the pile partial resistance factor for permanent and variable actions or operating situations ($\gamma_{R,Pp}$ = 1,50).

17.3.5 Scour

Sea floor scour shall be considered in the design of the foundation, as it affects both lateral and axial pile performance and capacity. Scour depends on very local circumstances and its prediction remains an uncertain art. Sediment transport studies can assist in defining scour design criteria, but local experience is the best guide. The uncertainty regarding design criteria should be taken into account by robust design and/or an operating strategy of monitoring and remediation as needed. Scour design criteria will usually be a combination of local and global scour and overall movement of the sea floor; see also ISO 19901-4.

17.4 Pile capacity for axial compression

17.4.1 General

Pile capacity for axial compression, as discussed in 17.4.2 to 17.4.5, relates to the axial resistance of a pile when the pile head is subjected to compressive actions along the pile axis. Pile capacity for axial tension is addressed in 17.5.

Further to the introductory discussion in 17.1.2, pile capacities are commonly determined using the simplified calculation model described in 17.4.2; the parameters that are used in this model are determined in accordance with 17.4.3 to 17.4.5. This simplified model has been developed and applied in many years of offshore practice and represents the current industry standard. However, the model does not provide any information about axial pile displacements which are important for serviceability requirements, especially in non-extreme conditions for actions due to permanent, variable and operating environmental actions that are generally well below the design actions. Axial pile behaviour aimed at meeting service requirements is referred to as *axial pile performance* and is discussed in 17.6. Methods for determining pile performance are described in A.17.6.2 and A.17.6.3.

The simplified model for pile capacity presented in 17.4.2 is based on a (quasi-)static and monotonic application of the axial actions and has no ability to reflect the complex occurrences that take place in the interaction between pile and soil during field conditions. To enhance understanding of the limitations of the model and to assist in applying engineering judgment to its results, it is useful to gain a better insight in the actual occurrences through the investigation of pile performance (see 17.6).

The relationships between mobilized axial shear transfer between pile and soil and the local pile displacement, and between mobilized end bearing resistance and the pile tip displacement can be determined using 17.7.

17.4.2 Representative axial pile capacity

The representative value of the axial capacity of piles, Q_{r}, including belled piles, shall be determined using Equation (17.4-1):

$$Q_{\mathrm{r}} = Q_{\mathrm{f}} + Q_{\mathrm{p}} = f \times A_{\mathrm{s}} + q \times A_{\mathrm{p}} \qquad (17.4\text{-}1)$$

where

Q_{f} is the representative value of the total skin friction resistance, in force units;

Q_{p} is the representative value of the end bearing capacity, in force units;

f is the unit skin friction, in stress units;

A_{s} is the side surface area of the pile;

q is the unit end bearing, in stress units;

A_{p} is the gross end area of the pile.

For open-ended pipe piles, the total end bearing capacity, Q_p, shall not exceed the sum of the end bearing capacity of the internal plug and the end bearing on the pile wall annulus. In computing the design actions in compression on the pile, the weight of the pile shall be considered.

In determining the capacity of a pile, consideration shall be given to the relative deformations between the soil and the pile as well as to the compressibility of the soil-pile system. In some circumstances, a more explicit consideration of axial pile performance effects on pile capacity is warranted. Additional discussion of these effects is given in 17.6 and A.17.6.

The foundation configurations should be based on those that experience has shown can be installed consistently, practically and economically under similar conditions with the pile size and installation equipment being used. Alternatives for possible remedial action in the event that design objectives cannot be obtained during installation should be investigated and defined prior to construction.

In the case of belled piles, the skin friction values on the pile section should be those given in 17.4 and 17.5. Skin friction on the upper bell surface and, possibly, on the pile for some distance above the bell should be discounted in computing the skin friction resistance, Q_f. The end bearing area of a pilot hole, if drilled, should also be discounted in computing the total bearing area of the bell.

17.4.3 Skin friction and end bearing in cohesive soils

There are a number of methods for calculating the skin friction and end bearing in cohesive soils. The method described below has been developed and applied over many years and is the current industry standard. However, caution should be exercised in its application as there are many more variables which affect pile capacity than those included in the design equations. This matter is discussed below and in A.17.4.3. For pipe piles in cohesive soils, the unit skin friction, f, in stress units, at any point along the pile, can be calculated using Equation (17.4-2):

$$f = \alpha c_u \tag{17.4-2}$$

where

α is a dimensionless factor;

c_u is the undrained shear strength of the soil (in stress units) at the point in question.

The factor α can be determined from Equations (17.4-3 a) and (17.4-3 b):

$$\alpha = 0{,}5\psi^{-0{,}5} \qquad \text{for } \psi \leqslant 1{,}0 \tag{17.4-3 a}$$

$$\alpha = 0{,}5\psi^{-0{,}25} \qquad \text{for } \psi > 1{,}0 \tag{17.4-3 b}$$

where

$\alpha < 1{,}0$

$$\psi = c_u/p_0' \qquad \text{for the point in question} \tag{17.4-4}$$

p_0' is the effective overburden stress at the point in question.

A discussion of appropriate methods for determining the undrained shear strength, c_u, and effective overburden stress, p_0', including the effects of various sampling and testing procedures, is included in A.17.4.3. For underconsolidated clays (clays with excess pore pressures undergoing active consolidation), α can usually be taken as 1,0.

Due to the shortage of pile load tests in soils having c_u/p_0' ratios greater than three, Equation (17.4-3) should be applied with considerable care for high c_u/p_0' values. Similar judgment should be applied for deep penetrating piles in soils with high undrained shear strength. Low plasticity clays should be treated with particular caution, see A.17.4.3.

For very long piles, some reduction in capacity can be warranted, particularly where the skin friction degrades on continued displacement. This effect is discussed in more detail in A.17.4.3.

Alternative means of determining pile capacity that are based on sound engineering principles and are consistent with industry experience are permissible. A more detailed discussion of alternative prediction methods is included in A.17.4.3.

For end bearing of piles in cohesive soils, the unit end bearing, q, in stress units, shall be computed using Equation (17.4-5):

$$q = 9c_u \tag{17.4-5}$$

The skin friction, f, acts on both the inside and the outside of the pile. The total axial resistance for pile compression is the sum of the external skin friction, the end bearing on the pile wall annulus, and the total internal skin friction or the end bearing of the plug, whichever is the lesser. For piles considered to be plugged, the bearing pressure may be assumed to act over the entire cross-section of the pile. For unplugged piles, the bearing pressure acts on the pile wall annulus only. That a pile is considered plugged or unplugged shall be based on static calculations. A pile can be driven in an unplugged condition but behave as plugged under static actions.

Skin friction resistance and end bearing capacity computed on the basis of the requirements above represent long-term capacities. Axial capacity immediately after installation is usually lower, especially in underconsolidated to slightly overconsolidated clays. This is dependent on the development of excess pore pressure in the soil during installation and its subsequent dissipation with time. When the design actions are applied to a pile foundation shortly after installation, the capacity of a pile immediately after installation and the increase in capacity with time are important design considerations. More discussion on the soil–pile set-up behaviour is provided in A.17.4.3.

For piles driven in undersized drilled holes, piles jetted in place or piles drilled and grouted in place, the selection of skin friction values shall take into account the soil disturbance resulting from installation. In general, f shall not exceed values for driven piles; however, in some cases, for drilled and grouted piles in overconsolidated clay, f can exceed these values. In determining f for drilled and grouted piles, the strength of the soil-grout interface, including potential effects of drilling mud, shall be considered. A further check shall be made of the allowable bond stress between the pile steel and the grout as recommended in 17.2.3.

In layered soils, skin friction values, f, in the cohesive layers shall be as given by Equations (17.4-2) to (17.4-4). End bearing values for piles tipped in cohesive layers with adjacent weaker layers may be taken as given in Equation (17.4-5) provided that

a) the pile achieves a penetration of two to three diameters or more into the layer in question, and

b) the tip is approximately three diameters or more above the bottom of the layer to preclude punch through.

Where these distances are not achieved, some modification of the end bearing is usually necessary. Where adjacent layers are of comparable strength to the layer of interest, the proximity of the pile tip to the interface is not a concern.

17.4.4 Skin friction and end bearing in cohesionless soils

This subclause describes a simple method for assessing pile capacity in cohesionless soils; other, recent and more accurate methods for predicting pile capacity in cohesionless soils are presented in A.17.4.4. These other methods are based on direct correlations of pile unit friction and end bearing data with cone penetration test (CPT) results. CPT-based methods are considered to be fundamentally better and have shown statistically closer predictions of pile load test results than the simple method described in this subclause. The

CPT-based methods also cover a wider range of cohesionless soils. However, before these new methods can be recommended for routine design in preference to the simple method presented herein, more experience with them is required. CPT-based methods should only be applied by qualified engineers who are experienced in the interpretation of CPT data and understand the limitations and reliability of these methods. Following pile installation, pile driving (instrumentation) data can be used to give more confidence in predicted capacities.

For pipe piles in cohesionless soils, the unit skin friction at a given depth, f, in stress units, can be calculated by Equation (17.4-6):

$$f = \beta p_0' \tag{17.4-6}$$

where

β is the dimensionless skin friction factor;

p_0' is the effective overburden stress at the point in question.

In the absence of specific data, β values for open-ended pipe piles that are driven unplugged may be taken from Table 17.4-1. For driven full displacement piles (i.e. closed-ended or fully plugged open-ended piles) values of β may be assumed to be 25 % higher than those given in Table 17.4-1. For long piles, f does not necessarily increase linearly with the overburden pressure as implied by Equation (17.4-6). In such cases, it is appropriate to limit f to the values given in Table 17.4-1.

For end bearing of piles in cohesionless soils, the unit end bearing, q, in stress units, may be computed using Equation (17.4-7):

$$q = N_q p_0' \tag{17.4-7}$$

where

p_0' is the effective overburden pressure at the pile tip;

N_q is the dimensionless bearing capacity factor.

Recommended values of N_q are presented in Table 17.4-1. For long piles, q does not necessarily increase linearly with the overburden pressure as implied by Equation (17.4-7). In such cases it is appropriate to limit q to the values given in Table 17.4-1. For plugged piles, the bearing pressure may be assumed to act over the entire cross-section of the pile. For unplugged piles, the bearing pressure acts on the pile annulus only. In this case, additional resistance is offered by friction between the soil plug and the inner pile wall. Whether a pile is considered to be plugged or unplugged shall be based on static calculations using a unit skin friction on the soil plug equal to the outer skin friction. Note that a pile can be driven in an unplugged condition, but can behave as plugged under static actions.

Load test data for piles in cohesionless soils indicate that variability in capacity predictions using the simple method described herein can exceed those for piles in clay. These data also indicate that the above method is conservative for short offshore piles (< 45 m, 150 ft) in dense to very dense sands loaded in compression and can be unconservative in all other conditions. Therefore, in unfamiliar situations, the designer should account for this uncertainty through the selection of conservative design parameters and/or higher safety factors. This is especially important where force redistributes after the development of maximum resistance occurs, which can lead to an abrupt (brittle) failure — such as is the case for short piles in tension.

Table 17.4-1 — Design parameters for cohesionless siliceous soil

Relative density[a]	Soil classification[b]	Skin friction factor β	Limiting unit skin friction values f kPa (kips/ft^2)	End bearing factor N_q	Limiting unit end bearing values q MPa (kips/ft^2)
Very loose Loose Loose Medium dense Dense	Sand Sand Sand-silt[c] Silt Silt	Not applicable[d]	Not applicable[d]	Not applicable[d]	Not applicable[d]
Medium dense	Sand-silt[c]	0,29	67 (1,4)	12	3 (60)
Medium dense Dense	Sand Sand-silt[c]	0,37	81 (1,7)	20	5 (100)
Dense Very dense	Sand Sand-silt[c]	0,46	96 (2,0)	40	10 (200)
Very dense	Sand	0,56	115 (2,4)	50	12 (250)

The parameters listed in this table are intended as guidelines only. Where detailed information such as *in situ* CPT records, strength tests on high quality samples, model tests or pile driving performance is available, other values are justified. Design values relate to the mid-point in each range of relative density.

[a] The following soil definitions for relative density descriptions are applicable:

Soil description	Relative density (%)
Very loose	0–15
Loose	15–35
Medium dense	35–65
Dense	65–85
Very dense	85–100

[b] Soil classifications are according to ISO 19901-4.

[c] Sand-silt includes soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

[d] Design parameters for these relative density/soil description combinations, given in previous API RP2A documents (see the Bibliography), can be unconservative. Hence CPT-based methods should be used for these soils (see A.17.4.4).

For soils that do not fall within the ranges of relative density and soil description given in Table 17.4-1, or for materials with unusually weak grains or compressible structure, Table 17.4-1 is not necessarily appropriate for selection of design parameters. For example, very loose soils or soils containing large amounts of mica or volcanic grains require special laboratory or field tests for selection of design parameters. Of particular importance are sands containing calcium carbonate (see A.17.4.4), which are found extensively in many areas of the ocean.

For piles driven in undersized drilled or jetted holes in cohesionless soils, the values of f and q should be determined by some reliable method that accounts for the amount of soil disturbance due to installation, but they shall not exceed the values for driven piles.

In layered soils, skin friction values, f, in the cohesionless layers shall be as given in Table 17.4-1. End bearing values for piles tipped in cohesionless layers with adjacent soft layers may also be taken from Table 17.4-1, provided that

a) the pile achieves a penetration of two to three diameters or more into the cohesionless layer, and

b) the tip is approximately three diameters or more above the bottom of the layer to preclude punch-through.

Where these distances are not achieved, some modification of the tabulated values is usually necessary. Where adjacent layers are of comparable strength to the layer of interest, the proximity of the pile tip to the layer interface is not a concern.

17.4.5 Skin friction and end bearing of grouted piles in rock

The unit skin friction of grouted piles in jetted or drilled holes in rock shall not exceed half the uniaxial compressive strength of the rock or grout, but in general should be much less than this value. The reduction depends on pile construction factors (such as roughness on the side of the hole) and on rock mass factors (such as the presence of discontinuities). The sidewall of the hole can develop a layer of slaked mud or clay, which will never gain the strength of the rock. The bond stress between the pile and the grout shall be checked in accordance with 17.2.3.

The end bearing capacity of the rock shall not exceed the uniaxial compressive strength of the rock or grout multiplied by a bearing capacity factor appropriate for the type of rock. In general, the end bearing capacity should be much less or ignored in the design, depending on pile construction factors, such as degree of removal of drill cuttings from the base of the hole, and on rock mass factors, such as the presence of discontinuities within the rock mass. The limiting end bearing capacity for this type of pile can be governed by stresses in the grout or in the pile steel.

Design values for (static) unit skin friction and end bearing can be found in various publications. It is noted that most publications on this subject refer to relatively "stubby" stiff piles as used in onshore practice (bored piles). Owing to the brittle response applicable to unit skin friction, design values given in these publications can be unconservative for long flexible piles as used in offshore practice. In addition, consideration shall be given to the fact that cyclic actions will adversely affect the axial capacity of such piles.

17.5 Pile capacity for axial tension

The ultimate pile pullout capacity is less than or equal to, but shall not exceed, Q_f, the total skin friction resistance. In computing the tensile design action on the pile, the weight of the pile shall be considered; the weight of the soil plug may also be considered, if this can be justified. For cohesive soils, f shall be the same as stated in 17.4.3. For cohesionless soils, f shall be computed according to 17.4.4. For rock, f shall be the same as stated in 17.4.5.

17.6 Axial pile performance

17.6.1 General

The axial capacity of a pile is its axial resistance, while pile performance refers to a specified service requirement by the owner (e.g. deflection(s) at the pile head). Both axial capacity and pile performance are dependent upon many variables (e.g. the types of soils, the pile characteristics, the installation methods, and the characteristics of the applied actions). The influence of these variables should be considered in pile design.

17.6.2 Static axial behaviour of piles

Pile axial deflections should be within acceptable serviceability limits and these deflections shall be compatible with the internal forces and movements of the structure. Axial pile behaviour is affected by directions, types, rates and sequence of the applied actions, by the installation technique, by soil type, by axial pile stiffness, as well as by other parameters; see A.17.6.2.

17.6.3 Cyclic axial behaviour of piles

Unusual conditions of actions on piles or limitations on design penetrations of piles warrant detailed consideration of cyclic effects.

Cyclic actions (including inertial actions) due to environmental conditions such as storm waves and earthquakes can have two potentially counteractive effects on the static axial capacity. Repetitive actions can cause a temporary or permanent decrease in resistance and/or an accumulation of deformation. Rapidly applied actions can cause an increase in resistance and/or stiffness of the pile. Very slowly applied actions can cause a decrease in resistance and/or stiffness of the pile. The resultant influence of cyclic actions will be a function of the combined effects of the magnitudes, cycles and rates of change of applied actions, the structural characteristics of the pile and the types of soils, see A.17.6.3.

The design pile penetration shall be sufficient to develop the design pile capacity, as discussed in 17.3 and 17.4, to resist the design static and cyclic actions. The design pile penetration can be determined by performing analyses of the pile-soil system subjected to static and cyclic actions. Analytical and numerical methods to perform such analyses are described in Annex A. The pile–soil, resistance–displacement, $t–z$ and $Q–z$, characterizations are discussed in 17.7.

17.6.4 Overall axial behaviour of piles

When any of the effects discussed in 17.6.3 and A.17.6.3 are explicitly considered in a pile analysis, the design static and cyclic actions shall be imposed on the pile top and the displacements of the pile determined. At the completion of the design actions, the maximum pile resistance and displacement shall be determined. Pile deformations shall meet structure serviceability requirements. The total pile resistance after the design actions shall meet the requirements of 17.3.4.

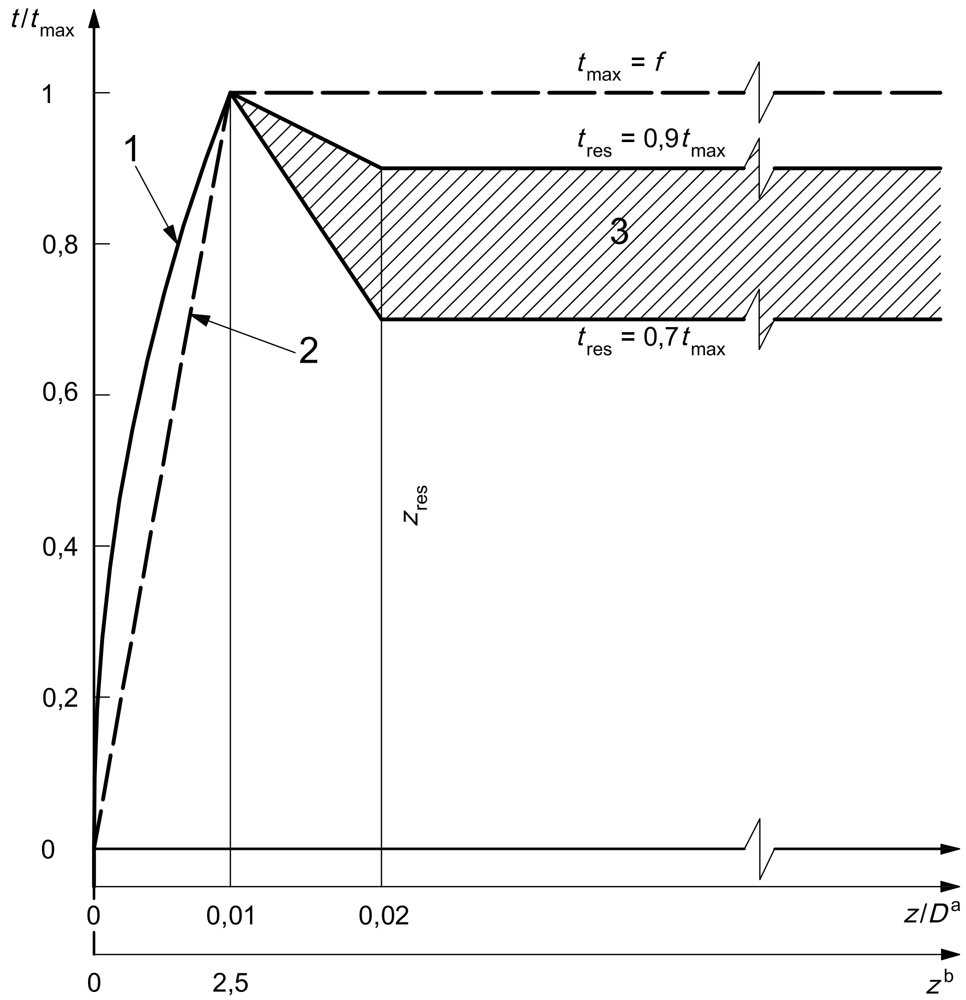
17.7 Soil reaction for piles under axial compression

17.7.1 General

The pile foundation shall be designed to resist the static and cyclic axial actions. The axial resistance of the soil for pile compression is provided by a combination of axial soil-pile adhesion and associated shear transfer along the sides of the pile, and end bearing resistance at the pile tip. The relationship between mobilized soil-pile shear transfer and local pile displacement at any depth is described using a t-z curve. Similarly, the relationship between mobilized end bearing resistance and axial tip displacement is described using a Q-z curve.

17.7.2 Axial shear transfer t-z curves

Various empirical and theoretical methods are available for developing curves for axial shear transfer and pile displacement, $t–z$ curves; these are referenced in A.17.7.2. Curves developed from pile load tests in representative soil profiles or based on laboratory soil tests that model pile installation can also be justified. In the absence of more definitive criteria, the following $t–z$ curves are recommended for non-carbonate soils. These recommended curves are shown in Figure 17.7-1.



Key

z local pile displacement (mm)

D pile diameter

t mobilized soil-pile adhesion (stress units)

t_{max} maximum soil-pile adhesion, or unit skin friction capacity, f, computed according to 17.4 (stress units)

t_{res} residual soil-pile adhesion

z_{res} axial pile displacement at which the residual soil-pile adhesion is reached

[a] z/D values, curve 1 and range 3 refer to behaviour in clays.

[b] z values and curve 2 refer to behaviour in sand.

Clays	
z/D	t/t_{max}
0,001 6	0,30
0,003 1	0,50
0,005 7	0,75
0,008 0	0,90
0,010 0	1,00
0,020 0	0,70 to 0,90
∞	0,70 to 0,90

Sands		
z (mm)	z (in)	t/t_{max}
0	0	0,00
2,5	0,100	1,00
∞	∞	1,00

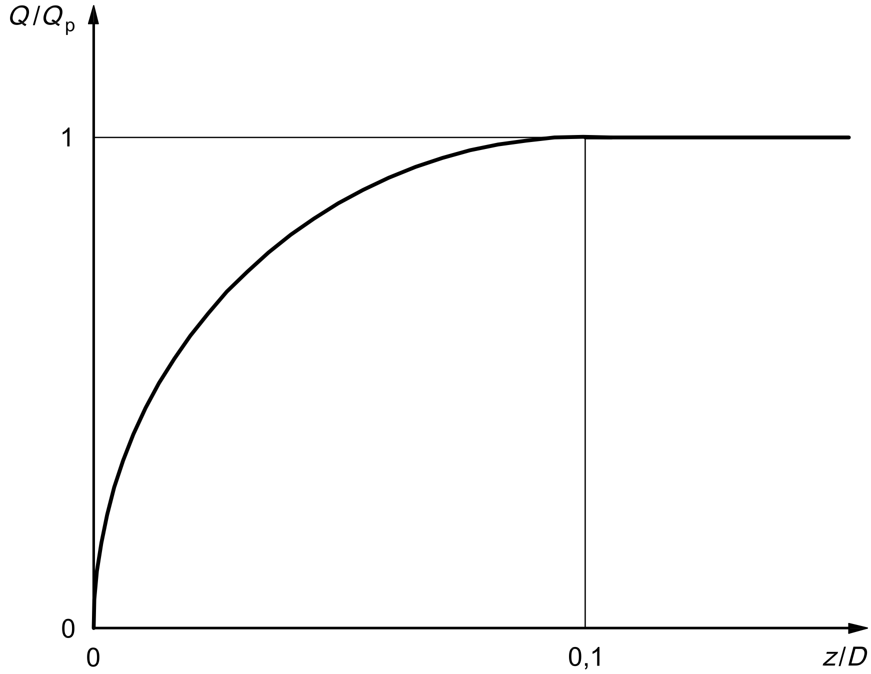
Figure 17.7-1 — Typical axial pile friction transfer-displacement curves

The shape of the t–z curve at displacements greater than that at which t_{max} is reached, as shown in Figure 17.7-1, should be carefully considered. Values of the residual adhesion ratio, t_{res}/t_{max}, and the axial pile displacement, z_{res}, at which it occurs are a function of soil stress-strain behaviour, stress history, pile installation method, sequence of pile action application and other factors.

The value of t_{res}/t_{max} for clays can range from 0,70 to 0,90. Laboratory, *in situ* or model pile tests can provide valuable information for determining values of t_{res}/t_{max} and z_{res} for various soils.

17.7.3 End bearing resistance-displacement, Q–z, curve

The representative end bearing capacity shall be determined as described in 17.4. However, relatively large pile tip displacements are required to mobilize the full end bearing resistance. A pile tip displacement of up to 10 % of the pile diameter can be required for full mobilization in both sand and clay soils. In the absence of more definitive criteria, the curve shown in Figure 17.7-2 is recommended for both sands and clays.



Key

z axial tip displacement

D pile diameter

Q mobilized end bearing resistance (force units)

Q_p representative value of end bearing capacity computed according to 17.4 (force units)

z/D	Q/Q_p
0,002	0,25
0,013	0,50
0,042	0,75
0,073	0,90
0,100	1,00
∞	1,00

Figure 17.7-2 — Pile end bearing resistance-displacement curve

17.8 Soil reaction for piles under lateral actions

17.8.1 General

The pile foundation shall be designed to resist static and cyclic lateral actions. The lateral resistance of the soil near the surface is significant to pile design, and the possible effects of scour on this resistance shall be considered. Estimates of these effects are discussed in A.17.8. In the absence of more definitive criteria, the procedures given in 17.8.2 to 17.8.7 may be used for constructing the relationships between lateral soil resistance and lateral displacement, p–y curves.

17.8.2 Representative lateral capacity for soft clay

For static lateral actions, the representative unit lateral capacity, p_r, of soft clay, in units of force per unit length, has been found to vary between $8c_u \cdot D$ and $12c_u \cdot D$, except at shallow depths where failure occurs in a different mode due to low overburden stress. Cyclic actions cause deterioration of lateral capacity below that for static actions. In the absence of more definitive criteria, the following representative value of the lateral capacity shall be used:

p_r increases from $3 \cdot c_u \cdot D$ to $9 \cdot c_u \cdot D$ as X increases from 0 to X_R according to Equation (17.8-1):

$$p_r = 3 \cdot c_u \cdot D = p_0' D + J c_u X \tag{17.8-1}$$

but p_r is limited by Equation (17.8-2):

$$p_r = 9 \cdot c_u \cdot D \text{ for } X > X_R \tag{17.8-2}$$

where

p_r is the representative lateral capacity, in units of force per unit length;

c_u is the undrained shear strength of undisturbed clay soil samples, in stress units;

D is the pile diameter;

p_0' is the effective overburden stress at depth, X;

J is a dimensionless empirical constant with values ranging from 0,25 to 0,5 having been determined by field testing, common practice being to use 0,5 if no other information is available;

X is the depth below the sea floor;

X_R is the depth below the sea floor to the bottom of a reduced resistance zone for uniform soils

$$X_R = \frac{6 \cdot c_u \cdot D}{\gamma' \cdot D + J \cdot c_u} \tag{17.8-3}$$

γ' is the submerged unit weight of soil in weight density units.

For non-uniform soils, Equations (17.8-1) and (17.8-2) can be solved by plotting the two equations for p_r vs. depth. The point of first intersection of the two equations is taken to be X_R. In general, X_R is in excess of 2,5 pile diameters. These empirical relationships do not necessarily apply where strength variations are erratic. These equations also do not apply in case of scour. Guidance for scour conditions is given in A.17.8.

17.8.3 Lateral soil resistance–displacement p–y curves for soft clay

Lateral soil resistance-displacement relationships for piles in soft clay are generally non-linear. The p–y curves for short-term static actions may be generated from Table 17.8-1. For the case where equilibrium has been reached under cyclic actions, the p–y curves may be generated from Table 17.8-2.

Table 17.8-1 — Mobilized lateral resistance–displacement data for short-term static actions

p/p_r	y/y_c
0,00	0,0
0,23	0,1
0,33	0,3
0,50	1,0
0,72	3,0
1,00	8,0
1,00	∞

p_r representative lateral capacity, units of force per unit length
p mobilized lateral resistance, units of force per unit length
y local lateral displacement
y_c $= 2{,}5\varepsilon_c D$
D pile diameter
ε_c strain at one-half the maximum deviator stress in laboratory undrained compression tests of undisturbed soil samples

Table 17.8-2 — Mobilized lateral resistance–displacement data for equilibrium conditions of cyclic actions

$X > X_R$		$X < X_R$	
p/p_r	y/y_c	p/p_r	y/y_c
0	0	0	0
0,23	0,1	0,23	0,1
0,33	0,3	0,33	0,3
0,50	1,0	0,50	1,0
0,72	3,0	0,72	3,0
0,72	∞	0,72 X/X_R	15,0
		0,72 X/X_R	∞

X depth below sea floor
X_R depth below the sea floor to bottom of reduced resistance zone for uniform soils [see Equation (17.8-3)]
p_r representative lateral capacity, units of force per unit length
p mobilized lateral resistance, units of force per unit length
y local lateral displacement
y_c $= 2{,}5\varepsilon_c D$
D pile diameter
ε_c strain at one-half maximum deviator stress in laboratory undrained compression tests of undisturbed soil samples

17.8.4 Representative lateral capacity for stiff clay

For static lateral actions the representative unit lateral capacity, p_r, of stiff clay [$c > 96$ kPa (2 kips/ft^2)] is similar to that for soft clay. However, due to rapid deterioration under cyclic actions, the representative lateral capacity shall be reduced for cyclic design considerations.

17.8.5 Lateral soil resistance–displacement p–y curves for stiff clay

While stiff clays also have non-linear stress-strain relationships, they are generally more brittle than soft clays. In developing stress–strain curves and subsequent p–y curves for cyclic actions, consideration shall be given to the possible rapid deterioration of lateral capacity at large displacements for stiff clays.

17.8.6 Representative lateral capacity for sand

For static lateral actions, the representative lateral capacity, p_r, for sand has been found to vary from a value at shallow depths determined by Equation (17.8-4) to a value at deep depths determined by Equation (17.8-5). At a given depth, the equation giving the smallest value of p_r shall be used as the representative capacity. These equations can be unconservative for layered soil conditions when the sand is overlain by soft clay.

$$p_{rs} = (C_1 X + C_2 D)\gamma' X \quad (17.8\text{-}4)$$

$$p_{rd} = C_3 D\, \gamma' X \quad (17.8\text{-}5)$$

where

s signifies shallow;

d signifies deep;

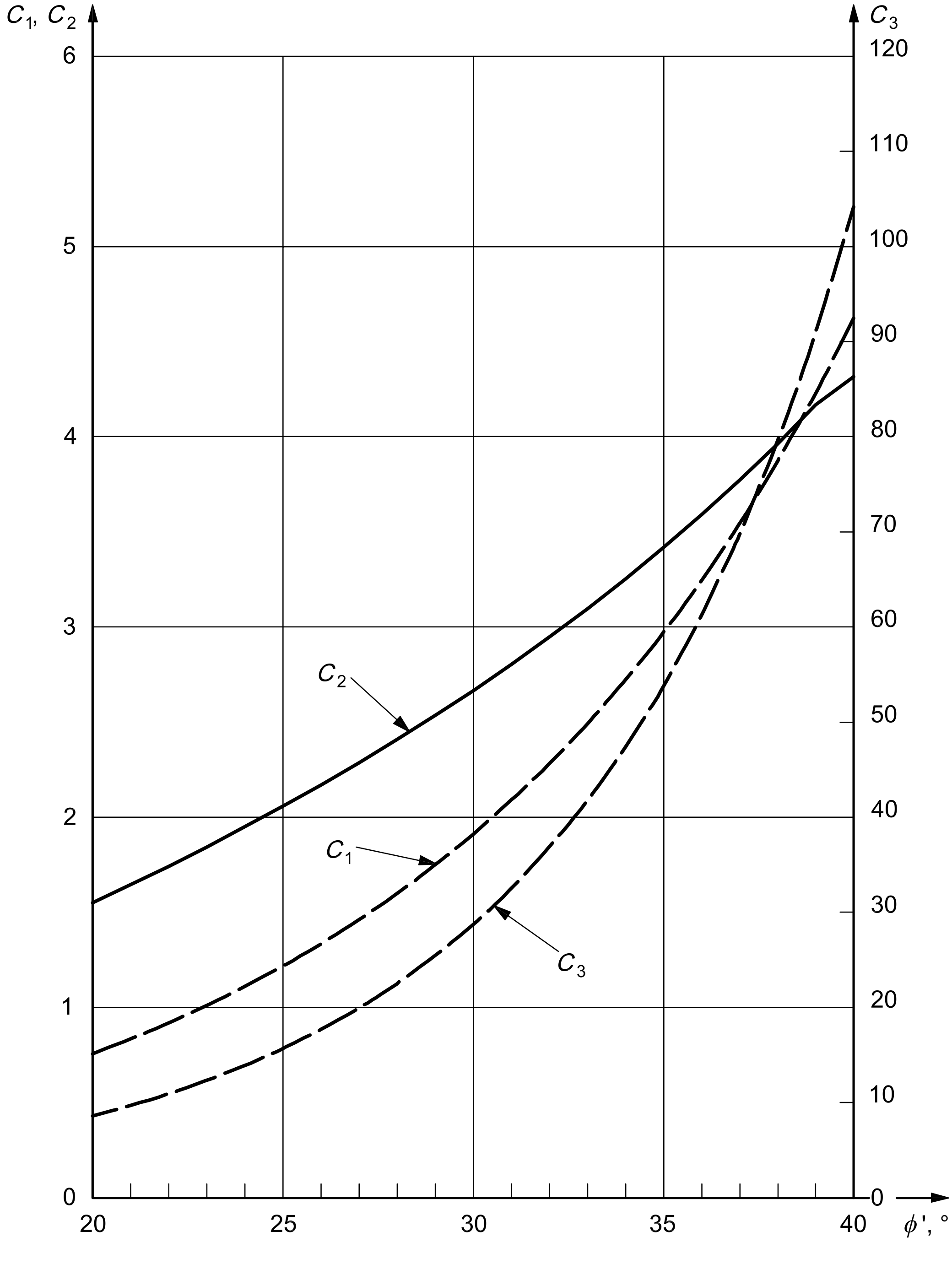
p_r is the representative lateral capacity in units of force per unit length;

γ' is the submerged unit weight of soil;

X is the depth below the sea floor;

C_1, C_2, C_3 are dimensionless coefficients determined from Figure 17.8-1 as a function of the angle of internal friction in sand, ϕ';

D is the pile diameter.



Key

ϕ' angle of internal friction

C_1, C_2, C_3 coefficients for lateral capacity

Figure 17.8-1 — Representative lateral capacity coefficients for sand

17.8.7 Lateral soil resistance–displacement p–y curves for sand

The lateral soil resistance-displacement p–y relationship for a pile in sand is also non-linear and in the absence of more definitive information may be approximated at any specific depth, by Equation (17.8-6):

$$p = A \cdot p_\mathrm{r} \cdot \tanh\left(\frac{k \cdot X}{A \cdot p_\mathrm{r}} \cdot y\right) \qquad (17.8\text{-}6)$$

where

A is a factor to account for static or cyclic actions, evaluated by

$A = \left(3{,}0 - \frac{0{,}8\ X}{D}\right) \geqslant 0{,}9$ for static actions, and

$A = 0{,}9$ for cyclic actions;

p_r is the representative lateral capacity at depth X in units of force per unit length;

k is the rate of increase with depth of the initial modulus of subgrade reaction, in units of force per volume, see Table 17.8-3;

X is the depth below the sea floor;

y is the lateral displacement at depth X.

The database for the lateral soil-pile behaviour in sands consists of free-head tests on piles in clean sands, with angles of internal friction ranging from 34° to 42°, as determined by shear box tests, drained triaxial tests or correlations with *in situ* tests.

Extrapolation of these data to soils outside the limits of experience, particularly to those sands with angles of internal friction less than 30°, should be done with caution. In particular, laboratory test results on such soils should be critically reviewed for evidence of anomalous behaviour and for the presence of considerable fractions of cohesive soils, either of which could require a different formulation for the p–y relationships.

In the absence of more definitive information, the values of the rate of increase with depth of the initial modulus of subgrade reaction k given in Table 17.8-3 are recommended.

Table 17.8-3 — Rate of increase with depth of initial modulus of subgrade reaction

ϕ'	k	
	MN/m^3	(lb/in^3)
25°	5,4	(20)
30°	11	(40)
35°	22	(80)
40°	45	(165)

17.9 Pile group behaviour

17.9.1 General

Consideration shall be given to the effects of closely spaced adjacent piles on the resistance-displacement characteristics of the pile group. Generally, for pile spacings less than eight diameters, group effects have to be evaluated.

17.9.2 Axial behaviour

For piles embedded in clays, the group capacity can be less than the single isolated pile capacity multiplied by the number of piles in the group; conversely, for piles embedded in sands, the group capacity can be higher than the sum of the capacities of the isolated piles. The group settlement in either clay or sand is normally larger than that of a single pile subjected to the average action per pile of the pile group.

17.9.3 Lateral behaviour

For piles with the same pile head fixity conditions and which are embedded in either cohesive or cohesionless soils, the pile group normally experiences greater lateral displacements than those undergone by a single pile subjected to the average action per pile of the corresponding group. The major factors influencing the group displacements and distribution of actions over the piles are the pile spacing, the ratio of the pile penetration to the pile diameter, the pile flexibility relative to the soil, the dimensions of the group, and the variations in the shear strength and stiffness modulus of the soil with depth.

17.9.4 Pile group stiffness and structure dynamics

When the dynamic behaviour of a structure is sensitive to variations in foundation stiffness, parametric analyses should be performed to bound the vertical and lateral foundation stiffness values to be used in the dynamic structural analyses; see A.17.9.4.

17.9.5 Resistance factors

The pile group capacity shall comply with the requirements of 17.3.4. Where there is a non-uniform distribution of actions into the piles, the partial action factors for individual piles in the group may be less than those specified in Clauses 9 to 11, provided that it can be demonstrated that the displacements and corresponding deformations of the piles and associated structural members are acceptable.

17.10 Pile wall thickness

17.10.1 General

The wall thickness of the pile may vary along its length and can be controlled at a particular point by any one of several load cases or requirements that are discussed in 17.10.2 to 17.10.9. The designer shall note the pile hammers that have been evaluated for use during driving on the installation drawings or specifications.

17.10.2 Pile stresses

Pile stresses due to the design action(s) shall be checked in accordance with Clause 13. A rational analysis considering the restraints placed upon the pile by the structure and by the soil shall be used to check the pile stresses for the portion of the pile that is not laterally restrained by the soil. General column buckling of the portion of the pile below the sea floor need not be considered, unless the pile is believed to be laterally unsupported because of extremely low soil shear strengths, large computed lateral deflections, or for some other reason.

Pile stresses and, as a result thereof, the pile wall thickness in the vicinity of the sea floor and possibly at other points are normally controlled by the combined axial force and bending moment which result from the design action(s) on the structure. The moment curve for the pile can be computed with soil resistance determined in accordance with 17.8, giving consideration to possible soil removal due to scour.

17.10.3 Pile design checks

When lateral deflections associated with cyclic actions at or near the sea floor are relatively large (e.g. exceeding y_{c} as defined in 17.8.3 for soft clay), consideration shall be given to reducing or neglecting the soil-pile adhesion through this zone.

17.10.4 Check for load case due to weight of hammer during hammer placement

Each pile or well conductor section on which a hammer (or pile top drilling rig, etc.) is placed shall be checked for the actions associated with placing the equipment. This load case can be the limiting factor in establishing the maximum length of add-on sections. This is particularly so in cases where piles are driven or drilled under an angle. The action effects that shall be checked include static bending, axial forces and lateral forces generated during initial hammer placement.

Experience indicates that reasonable protection from failure of the pipe wall due to the above load case is provided if the static strength is calculated as follows.

a) The projecting add-on section is considered as a free-standing fixed-end column with its appropriate effective length factor K, see A.17.10.4.

b) Axial forces and bending moments are calculated using the full factored weight of the hammer, pile cap and leads, acting through the centre of gravity of their combined masses with $\gamma_{f,G} = 1{,}3$ or $\gamma_{f,Q} = 1{,}5$, depending on how well the weight of each item is known, and the factored weight of the add-on section with $\gamma_{f,G} = 1{,}3$, taking into account the angle and eccentricities of the centres of mass. Nearly vertical add-ons are considered as inclined cantilevers, having an initial or realistic small out-of-plumb inclination of at least 2 % when determining their design moment.

c) The secondary bending moment, also to be determined, is the sum of the P-Δ moments due to the first-order lateral deflections of the add-on (considered as a fixed end cantilever). The deflection at the top should be associated with the factored weight of the hammer, etc. and the deflection at mid-height of the add-on should be associated with the factored weight of the add-on.

d) In accordance with 13.3.3, the beam-column resistance check shall be satisfied in accordance with Equation (13.3-3), but the bending stresses should include the secondary bending stresses due to deflection of the add-on. If the equation is applied to the plane in which bending takes place, the equation can be simplified to

$$\frac{\gamma_{R,c}\,\sigma_c}{f_c} + \frac{\gamma_{R,b}\,\sigma_b}{f_b}\left(1 + \frac{\Sigma P \times \Delta}{M}\right) \leqslant 1{,}0 \qquad (17.10\text{-}1)$$

where

$\Sigma P \times \Delta$ is the sum of the secondary P-Δ moments discussed in item b) above, where P is the factored weight of the item and Δ is the associated first order lateral deflection;

M is the primary bending moment at the foot of the add-on (in undeflected state);

σ_b is the primary bending stress due to M at the foot of the add-on;

$\gamma_{R,c}$, σ_c, f_c, $\gamma_{R,b}$ and f_b are as defined in Clause 13.

17.10.5 Stresses during driving

Consideration shall be given to the stresses that occur in the free-standing pile section during driving. Generally, stresses are checked against the conservative criterion that the sum of the stresses due to the impact of the hammer (the dynamic stresses) and the stresses due to axial force and bending moment(s) (the static stresses) shall not exceed the specified minimum yield strength (SMYS) of the steel. Less conservative criteria are permitted, provided that these are supported by sound engineering analyses and empirical evidence. The potential fatigue damage due to driving and its impact on the in-service fatigue life of the piles shall also be considered.

A method of analysis based on wave propagation theory should be used to determine the dynamic stresses (see 22.5.5). In general, it may be assumed that column buckling will not occur as a result of the dynamic portion of the driving stresses. The unfactored dynamic stresses should not exceed 80 % to 90 % of yield, depending on specific circumstances such as the location of the maximum stresses down the length of pile, the number of blows, previous experience with the pile-hammer combination and the confidence level in the analyses. Separate considerations apply when significant driving stresses are transmitted into the structure and damage to appurtenances shall be avoided.

When using hydraulic hammers, the driving energy can exceed the rated energy and this should be considered in the analyses. Also, the static stresses induced by hydraulic hammers need to be computed with special care due to the possible variations in driving configurations, for example, when driving vertical piles without lateral restraint that are exposed to environmental actions; see also 22.5.14.

17.10.6 Minimum wall thickness

The D/t ratio of the entire length of a pile shall be small enough to preclude local buckling at stresses up to the yield strength of the pile material. Consideration shall be given to the different situations and associated load cases occurring during the installation and the service life of a pile.

Experience indicates that for piles with a high D/t ratio, minor local damage near the pile tip (e.g. out-of-roundness or denting) can propagate during installation (e.g. by driving) to cause more extensive deformation and collapse of the pile. In addition to selecting an appropriate D/t ratio, the pile bottom section shall be checked for all load cases occurring during handling (e.g. pick-up, upending, stabbing, etc.) to ensure that local damage does not occur.

17.10.7 Allowance for underdrive and overdrive

With piles having thickened sections at the sea floor, consideration shall be given to providing an extra length of heavy wall material in the vicinity of the sea floor so that the pile will not be overstressed at this point if the design penetration is not reached. The amount of underdrive allowance provided in the design will depend on the degree of uncertainty regarding the penetration that can be obtained. In some instances, consideration shall also be given to providing an overdrive allowance in a similar manner, in the event that an expected bearing stratum is not encountered at the anticipated depth.

The possibility of underdrive and overdrive shall also be considered for determining the required length of the piles and for determining the position and extent of shear keys in grouted pile-to-sleeve connections.

17.10.8 Driving shoe

The purpose of driving shoes is to assist piles to penetrate through hard layers or to reduce driving resistances, thereby allowing greater penetrations to be achieved than would otherwise be the case. Different design considerations apply for each use.

If an internal driving shoe is provided for driving through a hard layer it should be designed to ensure that unacceptably high driving stresses do not occur at and above the transition point between the normal and the thickened section of the pile tip. It should also be checked that the shoe does not reduce the end bearing capacity of the soil plug below the value assumed in the design. External shoes are not normally used, as they tend to reduce the skin friction along the length of pile above them.

For reducing the internal skin friction during driving in cohesive soils, an internal driving shoe at the pile tip can be considered. The effect of an internal driving shoe should be taken into account when evaluating the total representative capacity of the pile.

17.10.9 Driving head

Any driving head at the top of the pile shall be designed in association with the installation contractor to ensure that it is fully compatible with the proposed installation procedures and equipment.

17.11 Length of pile sections

In selecting pile section lengths, consideration shall be given to

a) the capability of the lift equipment to raise, lower and stab the sections,

b) the capability of the lift equipment to place the pile driving hammer on the sections to be driven,

c) the possibility of a large amount of downward pile movement (e.g. due to self weight of the pile or immediately following the penetration of the closure of one of the structure’s legs, placement of the hammer or of a new add-on),

d) stresses developed in the pile section while lifting,

e) the wall thickness and material properties at field welds,

f) interference with the planned concurrent driving of neighbouring piles, and

g) the type of soil in which the pile tip is positioned during driving interruptions for field welding to attach additional sections.

In addition, static and dynamic stresses due to the hammer weight and operation shall be considered, as discussed in 17.10.4 and 17.10.5.

Each pile section on which driving is required shall contain a cut-off allowance to permit the removal of material damaged by the impact of the pile driving hammer. The normal allowance is 0,5 m to 1,5 m (2 ft to 5 ft) per section. Where possible, the cut for the removal of the cut-off allowance should be made at a conveniently accessible elevation.

17.12 Shallow foundations

17.12.1 General

The shallow foundations considered herein are typically used for the support of a piled offshore structure, e.g. mudmats, or the support of subsea structures. Detailed advice on the design of shallow foundations is contained in ISO 19901-4, while ISO 19903[6] and ISO 19905-1[4] provide special information for gravity base structures and jack-ups, respectively. It should be noted that the design practice in those International Standards applies material factors to soil properties, rather than a resistance factor to the capacity calculations, as is the case in Clause 17. Care should be taken when applying design procedures from one document to formulations in another.

The design of shallow foundations shall include, where appropriate to the intended application, consideration of the following:

a) stability, including failure due to overturning, bearing, sliding or combinations thereof;

b) static foundation deformations, including possible damage to components of the structure and its foundation or attached facilities;

c) dynamic foundation characteristics, including the influence of the foundation on action effects in the structure and the performance of the foundation itself under dynamic actions;

d) hydraulic instability, such as scour or piping action due to wave pressures, including the potential for foundation instability and damage to the structure;

e) installation and removal, including penetration and pull-out of shear skirts or the foundation base itself, and the effects of pressure build-up or draw-down of trapped water underneath the base;

f) flat-bottomed components or structures, which shall be designed with due consideration given to the possibility of non-uniform soil reactions which can cause hard spots acting on the base of the structure, and where under-base grouting, differential ballasting and/or site preparation may be used to mitigate the effects from such loads.

General recommendations pertaining to these aspects of shallow foundation design are given in ISO 19901-4. It should be noted that the complexity of the analyses can vary enormously from situation to situation. For example, a shallow circular foundation on a strong uniform soil under vertical actions only may be designed using simple techniques that can be found in most undergraduate soil mechanics text books. On the other hand, a mudmat of irregular shape on soft variable soil, subject to cyclic horizontal and vertical actions, could need a three-dimensional finite element analysis to produce an economical and reliable design.

17.12.2 Stability of shallow foundations

The equations to be considered in evaluating the stability of shallow foundations are given in ISO 19901-4. The equations are for static conditions (monotonic actions only). This is normally acceptable since a limiting weather criterion applies during the installation of offshore structures and, for subsea structures, design situations other than static are normally not applicable. However, if cyclic actions are likely to occur, e.g. during prolonged installation periods for structures with insufficient piles installed, these shall be taken into account separately.

18 Corrosion control

18.1 General

Fixed steel offshore structures require efficient control of corrosion to ensure that their strength is not reduced by progressive corrosion degradation. Corrosion damage can affect structural integrity and the ability to resist operational, environmental and accidental actions in various ways. One primary objective of corrosion control is to prevent such damage in fatigue-sensitive areas, as corrosion damage can result in stress concentrations for the initiation of fatigue cracks. Corrosion control will also avoid a potential reduction of the strength of structural components subjected to static actions only.

18.2 Corrosion zones and environmental parameters affecting corrosivity

Marine corrosion environments can be divided into the following corrosion zones:

a) atmospheric zone;

b) splash zone (or intermediate zone);

c) submerged zone;

d) buried zone.

The splash zone is defined in ISO 19900 as that area of the structure that is frequently wetted due to waves and tidal variations. However, surfaces which are only wetted during major storms are excluded. The extent of the splash zone is subject to large local variations and can vary approximately between 2 m and 10 m. In some areas, progressive subsidence of the structure shall be considered when defining the splash zone.

The atmospheric zone includes freely exposed and semi-sheltered areas above the splash zone. The sea water submerged zone extends below the splash zone and is defined here to include any sea water-flooded internal compartments. The buried zone includes any structural parts buried in the sea floor sediments or covered by disposed solids.

The corrosivity of corrosion zones varies as a function of geographical location. Of the primary environmental factors influencing corrosion, the main factor is temperature, which is also subject to significant seasonal variations. Salinity, sea currents, wave action and cycles of fresh water intrusion are other important parameters.

In the atmospheric zone, the frequency and duration of wetting (“time-of-wetness”) is a main factor affecting corrosion. The corrosive conditions are typically most severe in areas sheltered from direct rainfall and sunlight, but freely exposed to sea spray and condensation that facilitates accumulation of sea salts and moisture, with a resultant high time-of-wetness. A combination of high ambient temperature and time-of-wetness creates the most corrosive conditions.

In the upper submerged zone and the lower part of the splash zone, the corrosion environment is normally modified by marine growth. Depending on the type of growth and the local conditions, the net effect can be either to enhance or retard corrosion attack. Enhancement of corrosion processes by marine growth (e.g. through corrosive metabolites) is commonly referred to as microbiologically influenced corrosion (MIC). Marine growth can also interfere with systems for corrosion control, including coatings/linings and cathodic protection. Ice scoring in arctic waters can cause very high corrosion rates by removal of corrosion retarding rust layers, corrosion protective coatings, or marine growth. In deeper waters, variations in oxygen content and sea currents are the dominating parameters, although the corrosivity is relatively low.

In addition to freely exposed surfaces in the submerged zone, MIC can potentially occur in flooded steel members unless special precautions are taken to ensure that water remains sterile (e.g. by addition of a biocide).

In the buried zone, corrosion is predominantly related to MIC. In undisturbed sediments, MIC will only be significant in the uppermost layer. However, disturbance of sediments and discharge of drill-cuttings or other effluents can enhance bacterial activity and hence MIC.

NOTE Other splash zone definitions can apply in regulations or in other design codes.

18.3 Forms of corrosion, associated corrosion rates and corrosion damage

Corrosion damage to uncoated carbon steel in the atmospheric zone and in the splash zone is associated with oxygen attack and is typically more or less uniform. In typical conditions for the atmospheric zone, the steady state corrosion rate for carbon steel (i.e. as uniform attack) is normally around 0,1 mm/year or lower. In the splash zone and the most corrosive conditions for the atmospheric zone (i.e. high time-of-wetness and high ambient temperature), corrosion rates can amount to 0,3 mm/year, and for internally heated surfaces in the splash zone even much higher (up to an order of magnitude of 3 mm/year).

In the submerged and buried zones, corrosion is mostly governed by MIC causing colonies of corrosion pits. Welds are often preferentially attacked. Corrosion as uniform attack is unlikely to significantly exceed about 0,1 mm/yr but the rate of pitting can be much higher: 1 mm/yr and even more under conditions favouring high bacterial activity (e.g. ambient temperature of 20 °C to 40 °C and access to organic material).

In most cases, the static strength of large structural components is not jeopardized by MIC, due to its localized form. However, MIC can initiate fatigue cracking of components subjected to cyclic actions.

Aluminium alloys of the 5 000 and 6 000 series as used for topsides structural components are highly resistant to marine atmospheres and suffer only superficial staining or micro-pitting. However, galvanic coupling (i.e. metallic plus electrolytic coupling) to structural steel, stainless steels and copper alloys shall be avoided. Otherwise, severe galvanic corrosion of aluminium can result at the point of contact. Similarly, structural steel can suffer enhanced corrosion if in galvanic contact with stainless steel or with copper base alloys. The propensity for galvanic corrosion is highest in the submerged zone but is normally prevented by cathodic protection.

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

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

18.4 Design of corrosion control

18.4.1 General

The following main systems for corrosion control of fixed steel offshore structures apply:

a) coatings, linings and wrappings;

b) cathodic protection;

c) corrosion-resistant materials;

d) corrosion allowance.

18.4.2 Considerations in design of corrosion control

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

a) regulatory requirements;

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

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

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

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

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

18.4.3 Coatings, linings and wrappings

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

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

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

In the sea water submerged zone, organic coatings or thermally sprayed aluminium may be used to reduce weight or hydrodynamic actions due to drag caused by galvanic anodes and/or to improve the current distribution by cathodic protection. On subsea production systems, paint coatings are primarily used to ease visibility. In the upper submerged zone coatings may also be chosen to improve periodic removal of marine growth.

Coating and lining systems shall primarily be selected based on proven experience for a specific application and environment. Comprehensive field testing is required when practical experience is lacking. For the atmospheric and splash zones, maintainability is a major criterion, while compatibility with cathodic protection is normally required in the submerged and buried zones. Resistance to damage, e.g. due to the removal of marine fouling, is also required.

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

18.4.4 Cathodic protection

18.4.4.1 Cathodic protection systems

Cathodic protection may be effected using

— galvanic (sacrificial) anodes, or

— impressed current (IC) from one or more rectifiers.

Cathodic protection is applicable to the submerged and the buried zones. In the splash zone, cathodic protection is generally fully effective for any part of the structure extending below the lowest astronomical tide (LAT) and marginally effective in the tidal zone.

With adequate design, any form of corrosion damage can be prevented, including galvanic corrosion between dissimilar materials. Cathodic protection is also efficient in preventing corrosion damage in narrow crevices between structural components. It is generally accepted that cathodic protection restores resistance to initiation of fatigue cracks to the same level as in dry air. Laboratory testing has shown that cathodic protection can enhance propagation of fatigue cracks at high stress intensities. However, practical experience indicates that this effect is not critical to normal structural materials at the potential range required below.

Cathodic protection by galvanic anodes has proven reliability for long-term protection, even in the harshest weather conditions. IC systems are less tolerant of any shortcomings of design, installation and maintenance, and require a dependable external current source. IC systems have also been used for upgrading cathodic protection systems based on galvanic anodes. Reliability and costs for installation, operation and maintenance shall be taken into account when choosing between IC and galvanic anode systems.

Owing to the risk of hydrogen induced stress cracking (HISC), steels with a specified minimum yield strength (SMYS) in excess of 720 MPa shall not be used for critical cathodically protected components without special considerations. This imposes some restrictions on the subsea use of high strength parts, e.g. bolting. Furthermore, any welding (or other construction method affecting ductility or tensile properties) shall be carried out according to a qualified procedure which limits hardness to HV350. This restricts the use of welded structural steels to a maximum SMYS of approximately 550 MPa.

The cathodic protection system for fixed steel offshore structures shall account for risers, conductors, pull-tubes and other submerged objects electrically connected to the structure. In some cases, subsea pipelines are provided with electrical insulation to prevent interaction between the structure and pipeline cathodic protection systems.

Current drain to piles, well casings, concrete reinforcing bars or other components not considered to require cathodic protection shall be assessed.

Internal compartments flooded with sea water shall be provided with sufficient galvanic anodes, suitably arranged, to last the design service life of the structure. IC systems are not normally applicable for this purpose.

The design shall ensure a protection potential within the range $-0{,}80$ V to $-1{,}1$ V relative to Ag/AgCl/sea water. More negative potentials can be achieved by IC systems, but these can be harmful to coatings and promote HISC in ordinary structural steels.

The current demand, I_c, for cathodic protection of external surfaces in the submerged and buried zones, and for wetted surfaces in any internal compartments shall be calculated from Equation (18.4-1):

$$I_c = A_c \cdot k_c \cdot i_c \qquad (18.4\text{-}1)$$

where

I_c is the current demand (A);

A_c is the actual surface area (m^2);

k_c is the coating break-down factor for any coated surfaces, with $k_c = 1$ for bare steel;

i_c is the design current density (A/m^2).

I_c shall be calculated as the average current demand, $I_{c,average}$, to maintain cathodic protection throughout the design service life, t_L, in years, of the system. The initial and final current demands, i.e. $I_{c,initial}$ and $I_{c,final}$

respectively, required to polarize the relevant surfaces to a protection potential of – 0,80 V relative to Ag/AgCl/sea water, shall also be calculated.

Design current densities and coating break-down factors for calculations of average and initial/final current demands are given in the applicable cathodic protection design codes; see A.18.

For large structures it is always convenient, and often necessary, to subdivide the structure into units to be protected. The division may, for example, be based on depth zones. Each internal compartment shall comprise at least one unit for individual design.

The design life, t_L, in years, of cathodic protection systems shall normally be equal to the design service life of the structure itself. If the design is based on replacement of anodes, provisions to facilitate retrofitting shall be addressed during the design.

Possible interaction with other structures, pipelines, and vessels shall be evaluated. The need for temporary corrosion protection prior to the energizing of the IC system shall be assessed.

To ensure consistency, design parameters shall all be collected from the same cathodic protection design code selected, see A.18.

18.4.4.2 Galvanic anode systems

Based on the average total current demand, $I_{c,average}$, for each unit (including any current drain), the total net anode mass, m_{total} (kg) required to maintain cathodic protection throughout the design service life t_L (in years) is calculated from Equation (18.4-2):

$$m_{total} = \frac{I_{c,average} \cdot t_L \cdot 8\,760}{u \cdot \varepsilon} \tag{18.4-2}$$

where

u is the anode's utilization factor;

ε is the anode materials electrochemical efficiency (A·h/kg).

Design values for ε and u are, again, given in the applicable cathodic protection design codes; see A.18.

From the required total net anode mass, m_{total}, a tentative selection of anode dimensions and number of anodes can be determined. It shall subsequently be demonstrated that this selection meets the requirements for the initial/final current output, I_a (A), per anode, and the total current capacity, C_a (A·h).

The anode current output, I_a, is calculated from Ohm's law according to

$$I_a = \frac{E_c - E_a}{R_a} \tag{18.4-3}$$

where

E_c is the design protective potential (V) matching the initial/final design current densities (i.e. normally – 0,80 V relative to Ag/AgCl/sea water);

E_a is the design closed circuit anode potential (V);

R_a is the anode resistance (ohms).

E_c, E_a and R_a are also given in the applicable cathodic protection design codes; see A.18.

The current capacity c_a of one anode is given by:

$$c_a = m \cdot u \cdot \varepsilon \tag{18.4-4}$$

where m is the net mass per anode.

The total current capacity C_a for n anodes thus becomes:

$$C_a = n \cdot c_a \tag{18.4-5}$$

Anode dimensions and net weight shall be selected to match all requirements for current output (initial/final) and current capacity for a specific number of anodes. Calculations shall be carried out to demonstrate that the following requirements are met:

$$C_a = n \cdot c_a \geqslant I_{c,average} \cdot t_L \cdot 8\,760 \tag{18.4-6}$$

$$n \cdot I_{a,initial} \geqslant I_{c,initial} \tag{18.4-7}$$

$$n \cdot I_{a,final} \geqslant I_{c,final} \tag{18.4-8}$$

The final current output shall be calculated using the estimated anode resistance when the anode has been consumed to its utilization factor.

The calculated number of anodes shall be distributed to provide a uniform current distribution, taking into account the current demand of individual surface areas and any current drain.

Design of galvanic anode cores shall take into account forces induced by environmental actions and installation activities (including any pile driving operations). For large stand-off anodes, the use of doubler plates shall be considered. Effects of added weight, increased drag actions, and subsea operations shall be evaluated.

Anodes shall be located in the submerged zone for the protection of both splash zone, submerged zone and buried zone. The uppermost part of the submerged zone should be avoided, in order to reduce the effects of tide, waves, and marine fouling. Anodes shall not be located close to joints and other areas that are critical to structural integrity. The distribution of anodes shall otherwise reflect the current demand of individual areas to be protected.

For sacrificial anode systems, equipping a few anodes with instruments to enable current output monitoring can aid in future maintenance and retrofit decisions.

18.4.4.3 Impressed current systems

The design of impressed current (IC) cathodic protection systems shall include extra capacity (i.e. compared to the calculated current demand) to compensate for a more uneven current distribution from the relatively few, high output IC anodes. Furthermore, redundancy shall be included to compensate for some deficiency of individual anodes and rectifiers. The design shall include detailed procedures for maintenance (replacement) of anodes and other subsea equipment.

IC systems shall have a structure-to-sea water monitoring system that is able to show that cathodic protection is maintained within specified limits at both areas closest to and furthest from the anodes.

18.4.5 Corrosion-resistant materials

The selection of corrosion-resistant materials for structural components shall take into account their anticipated corrosion resistance for the intended application (including resistance to environmentally induced cracking) and compatibility with other materials, mechanical properties and ability to fabricate.

Precautions shall be taken to prevent galvanic corrosion of less resistant materials. In the submerged and buried zones, galvanic corrosion can be efficiently prevented by cathodic protection. However, in the atmospheric and splash zones, special precautions shall be taken to prevent galvanic corrosion. These can include coating of the component with the highest electrochemical potential or electric insulation.

NOTE The term “environmentally induced cracking”, sometimes also called “environmentally assisted cracking”, includes stress corrosion cracking, corrosion fatigue and hydrogen-induced cracking associated with hydrogen generated by cathodic protection.

18.4.6 Corrosion allowance

A corrosion allowance, i.e. extra steel thickness to compensate for the effect of metal loss by corrosion on structural integrity, is sometimes applicable, e.g. for tubular sections in the splash zone. It can be used alone or in combination with a coating. The thickness of any corrosion allowance shall be determined taking expected corrosivity, design service life and maintenance plans into account.

18.5 Fabrication and installation of corrosion control

18.5.1 General

Fabrication procedures can affect the corrosion resistance of materials, in particular for certain corrosion-resistant materials. All fabrication involving welding or brazing to structural components shall be performed in accordance with Clause 20, regulatory requirements, applicable codes/standards, and approved project specific procedures and drawings.

18.5.2 Coatings and linings

Recommendations for surface preparation, materials, coating application, inspection, and repairs are given in applicable standards and practices.

Quality control during surface preparation, coating application, and repairs is most essential to ensure consistent performance of coatings and linings. Follow-up of coating work, from surface preparation to final inspection by a certified coating inspector, is highly recommended.

18.5.3 Cathodic protection

Manufacturing and quality control of sacrificial anodes shall be performed in accordance with a procedure which defines compositional limits of anode and anode core materials, weights and dimensional tolerances, visual inspection and permissible surface defects, marking and documentation. Short-term electrochemical testing may be specified for the verification of electrochemical performance on a heat basis.

Installation of galvanic anodes by welding or brazing shall be in accordance with 18.5.1. After completing installation, anode locations and sizes shall be verified.

Commissioning of IC cathodic protection systems shall include detailed structure-to-sea water potential measurements for verifying readings from fixed reference electrodes and to confirm that the protective potential range is achieved for all concerned parts of the structure.

18.5.4 Corrosion-resistant materials

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

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

18.6.1 General

In-service inspection of corrosion control is a periodic activity for confirmation of the physical condition and integrity of the corrosion control system(s) and/or the actual components to be protected. Monitoring of corrosion control refers to regular recording of data associated with corrosion control. It is frequently carried out on-line.

EXAMPLE Recordings of structure-to-sea water potentials from fixed reference electrodes.

Planning of schemes for inspection and monitoring of corrosion control shall take into account

a) regulatory requirements,

b) the criticality of the overall system and of the individual components being protected,

c) the type and severity of the corrosion environment(s),

d) potential forms of corrosion damage,

e) the capability of inspection and monitoring tools, as well as accessibility for inspection, and

f) results and consequences of previous inspections and/or monitoring.

NOTE 1 See Clause 23 for in-service inspection and corrosion control requirements for structural integrity management.

NOTE 2 In other standards, other definitions of inspection, monitoring and maintenance of corrosion control can apply; for example, corrosion monitoring is sometimes defined to include corrosion inspection and vice versa.

18.6.2 Coatings and linings

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

18.6.3 Cathodic protection

Inspection of cathodic protection may include visual examination of anodes and measurements of structure-to-sea water potentials and anode-to-sea water potentials. A protection potential of – 0,80 V relative to Ag/AgCl/sea water or more negative is a generally accepted criterion for cathodic protection of carbon steel in sea water. Accurate and detailed potential measurements require a suitable reference electrode positioned by a diver or an underwater vehicle. A general assessment of the protection of the structure or parts thereof can be obtained by lowering a reference electrode from the surface.

Galvanic anode potential measurements shall be performed as part of a detailed structure survey. Average anode and cathode potentials can be used to estimate cathode current demand and anode service life.

Galvanic anode potential measurements shall be performed with minimum disruption of any deposit layers. Accurate estimates of anode consumption can require removal of deposits and should therefore be performed after potential measurements have been completed.

Monitoring of galvanic anode cathodic protection systems is sometimes performed using recordings from fixed reference electrodes.

For galvanic anode systems, an initial survey comprising structure-to-sea water potential measurements (with emphasis on the more critical areas) and preferably anode-to-sea water potential measurements should be performed within 1 year after installation. The frequency of further potential surveys shall be assessed on the basis of results from any such surveys performed earlier and from periodic visual inspection of structural components and anodes.

For IC cathodic protection systems, a comprehensive mapping of the potential distribution shall be performed during commissioning (see 18.5.3). Fixed reference electrodes used to control current output shall be calibrated at regular intervals; rectifier voltage and total current outputs shall be recorded on a frequent basis, together with readings from individual anodes and fixed reference electrodes.

Safety regulations can prohibit diver activities whilst IC systems are in operation. Divers can suffer electric shocks from faulty electric equipment or from direct contact with anodes. The following shall be considered when divers inspect structures with IC systems installed:

— when divers are not involved in or not aware of cathodic protection (such as for marine growth cleaning), it is recommended that the IC system be switched off during diving;

— for divers involved in cathodic protection inspection, a safe distance shall be kept between divers and IC anodes which are in operation;

— for the performance of close visual inspection of IC anodes, the current supply to anodes shall be switched off.

18.6.4 Corrosion-resistant materials

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

19 Materials

19.1 General

This clause presents good practice, supported by industry-wide consensus, for the selection of steels that are expected to perform effectively over the design service life of a structure, while allowing practical and economical fabrication and inspection.

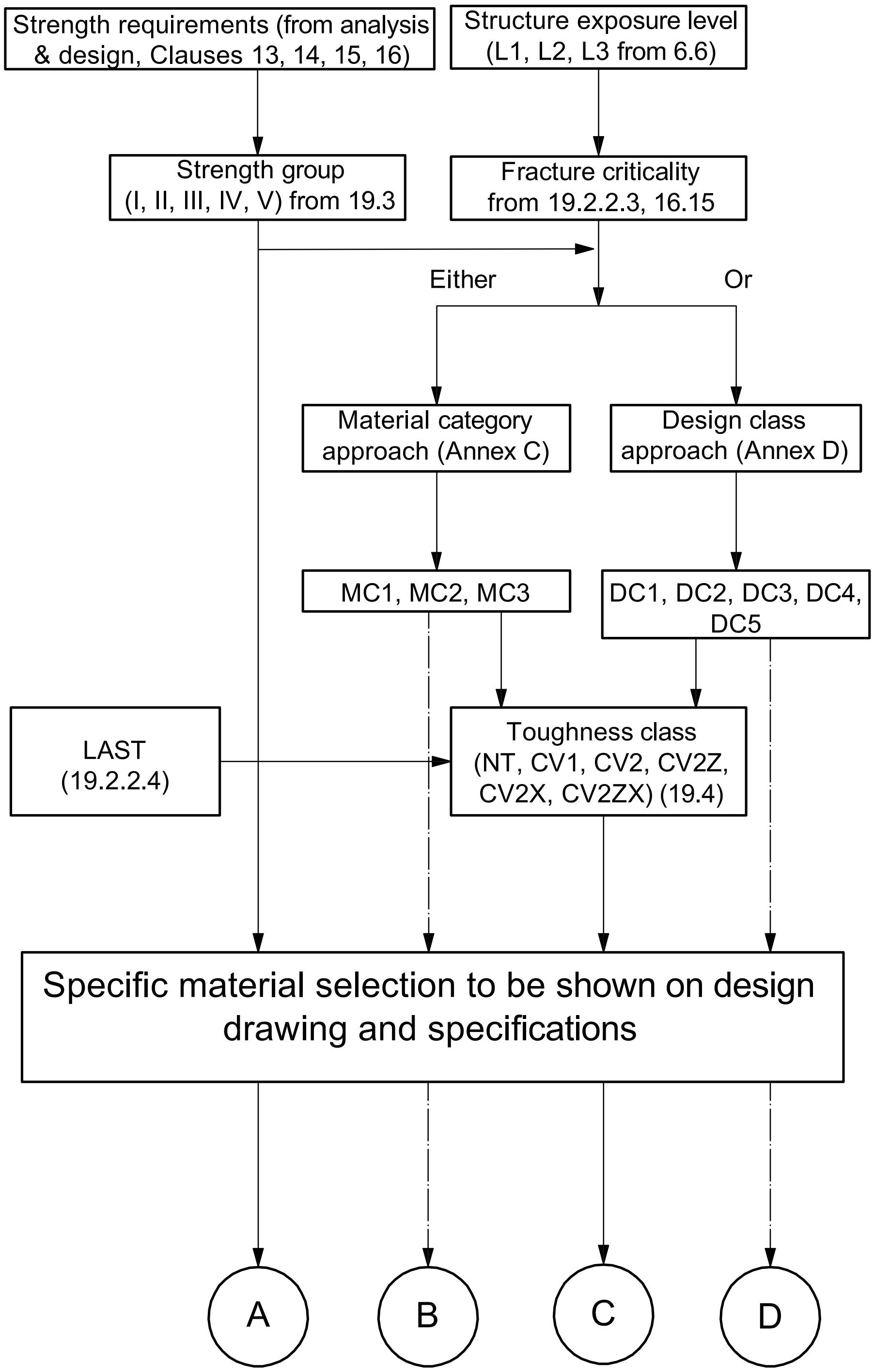
Two methods are presented for determining the particular steel specifications to be used for a specific structure and the accompanying welding, fabrication and inspection requirements. These methods, briefly introduced in 19.2.4 and 19.2.5 and described in detail in Annexes C and D, are generally referred to as

a) the material category (MC) approach, and

b) the design class (DC) approach

A flow chart of the process followed by this clause is given in Figure 19.1-1. The material category (MC) and design class (DC) methods are mutually exclusive. Once the method has been selected it is not interchangeable at any stage with the other. The objective is to guide the designer in arriving at specific material selections, which shall be shown on design drawings and in project specifications. The strength grades and toughness classes of the selected steels carry over to related welding, inspection, fabrication and quality requirements in Clause 20.

For any specific project application, the owner shall define the mutual responsibilities between design and fabrication.



A, B, C, D continue to Figure 20.1-1, as input to welding, inspection, fabrication and QA/QC documentation considerations.

Figure 19.1-1 — Flow chart for material selection

19.2 Design philosophy

19.2.1 Material characterization

For the purposes of material selection and utilization in offshore structures, steels are characterized as belonging to a strength group and a toughness class.

Strength groups (see 19.3) are defined by a range of yield strengths, which are determined by tensile testing of specimens in accordance with steel manufacturing specifications. The specified yield strength is a design criterion used to size members in accordance with selected design standards.

Toughness classes (see 19.4) are determined by the ability of steels to achieve a minimum Charpy V-Notch (CVN) test energy at a specified minimum temperature.

Toughness becomes more important as the magnitudes of varying actions increase, as the criticality of the structure increases and as service temperatures decrease. The toughness of the steel affects the fracture resistance.

The designer can also consider (at least qualitatively) the extent of plastic deformation needed before fracture, the degree of restraint as influenced by section thickness, as well as residual stress as influenced by weld shrinkage and general section thickness of the parts.

19.2.2 Material selection criteria

19.2.2.1 Yield strength requirements

The minimum yield strength for a given component geometry and thickness shall satisfy the strength requirements defined in Clauses 13, 14, 15 and 16, in conjunction with the actions described in Clauses 8, 9, 10 and 11.

19.2.2.2 Structure exposure level

The material selection shall take into account the structure's exposure level (L1, L2, L3), established at the outset of the design process in accordance with 6.6.

19.2.2.3 Component criticality

In the selection of the appropriate material, consideration shall be given to the criticality of the component. A component (member or joint) is considered critical if its sole failure is likely to lead to a catastrophic structural collapse (3.12). In general, the toughness requirements and inspection of this International Standard ensure that the material selected for a critical component has higher toughness and ductility requirements and more stringent inspection requirements than the material selected for a component with high redundancy, where alternative load paths in the structure lessen the risk of a major failure.

19.2.2.4 Lowest anticipated service temperature

The steel performance characteristics are affected by the lowest anticipated service temperature (LAST). The LAST value to be used in the material selection shall be in accordance with applicable regulatory requirements in the region of application (regional or national standards, regional information in Annex H, etc.). Suggested LAST values for certain offshore areas are given in A.19.2.2.4.

The LAST value establishes the temperature for CVN toughness, which shall be taken as the LAST minus the temperature margin shown in Table 19.4-1.

19.2.2.5 Other considerations

Consideration shall be given to other factors, such as resistance to fracture in the presence of plastic deformation, residual stress, thickness restraint, cost, availability, weldability, resistance to lamellar tearing, and demonstrated successful use. Guidance on these topics is generally included in complementary standards or guidelines that need to be used in conjunction with this International Standard.

19.2.3 Selection process

Once the criteria described in 19.2.2 have been established, the steel selection process follows the logical flowchart shown in Figure 19.1-1.

Key parts of the process are the two methods described in 19.2.4, the material category (MC) approach, and in 19.2.5, the design class (DC) approach, which have been very successfully used within the offshore industry for many years. Both methods address the issues described in 19.2.2, albeit from different philosophical perspectives.

Either approach provides a framework for material selection, welding qualification and extent of inspection commensurate with the level of reliability implicit in the design process. Details of the two methods are given in Annex C and Annex D respectively. Each annex provides normative details concerning the implementation of the procedures applicable to its particular method.

As the two methods are incompatible, the user shall not combine a partial implementation of the procedures relative to one approach with those of the other approach.

As an alternative to the MC and DC approaches, other rational procedures may be considered.

19.2.4 Material category approach

In the MC approach (Annex C) the steel selection is based on the interrelation of the structure's exposure level, the material yield strength and toughness, and the component consequence ranking. Thickness is considered in connection with component type. The corresponding welding and inspection requirements are given in Annex E.

19.2.5 Design class approach

The DC approach (Annex D) allows wider discretion in selecting the appropriate material strength group and toughness class on the basis of a component's criticality rating: DC 1 to DC 5, with DC 1 being the most critical. The corresponding welding and inspection requirements are given in Annex F.

19.3 Strength groups

Steels are grouped according to specified minimum yield strength (SMYS), as presented in Table 19.3-1.

Table 19.3-1 — Steel group SMYS requirements

Group	SMYS range
I	220 MPa (32 ksi) to 275 MPa (40 ksi)
II	> 275 MPa (40 ksi) to 395 MPa (57 ksi)
III	> 395 MPa (57 ksi) to 455 MPa (66 ksi)
IV	> 455 MPa (66 ksi) to 495 MPa (72 ksi)
V	> 495 MPa (72 ksi)

19.4 Toughness classes

This subclause describes the toughness classes used in this International Standard, and the types of steel for which they are considered applicable. All testing shall conform to an internationally recognized standard.

Class NT steels (not CVN tested) are considered suitable for application in critical welded components at service temperatures above 0 °C (32 °F).

Class CV1 steels are suitable for use where service temperatures, thickness, cold work, restraint, stress concentration, impact loading and/or lack of redundancy indicate the need for improved notch toughness. Class CV1 steels shall, as a minimum, exhibit CVN test energy as shown in Table 19.4-1.

Class CV2 steels have a large margin between CVN testing temperature and service temperature, as prescribed in Table 19.4-1. Such steels are suitable for major primary structures or structural components and for critical or non-redundant components, particularly in the presence of factors such as

— high stresses and stress concentrations,

— high residual stresses,

- severe cold work from fabrication,
- low temperatures,
- high calculated fatigue damage, and/or
- impact loading.

In addition to all the other characteristics of class CV2 steel, class CV2Z steels shall have through-thickness (short transverse direction) ductility for resistance to lamellar tearing caused by tensile stress in the direction of thickness. Through-thickness ductility shall be demonstrated by either having a minimum reduction in area of 30 % in a tension test conducted on a specimen cut from the through-thickness direction, or by specifying a sulphur content by weight (P_S) of 0,006 % or less in the ladle analysis.

Class CV2X steels shall have a formal CTOD pre-qualification in addition to the other characteristics of class CV2 steel.

Class CV2ZX steels shall have a formal CTOD pre-qualification in addition to the characteristics of class CV2Z steel.

Testing of material may be performed in accordance with suitable national or internationally recognized standards.

Minimum steel toughness requirements are given in Table 19.4-1. This table is not intended to supersede the designer's judgment for special situations, such as the need to increase the toughness level for any of the reasons described in 19.2.1.

Table 19.4-1 — Minimum toughness requirements for structural steels

Steel group	SMYS range MPa (ksi)	Charpy toughness	Toughness classes and Charpy impact test temperature			
			NT (CVN testing not required)	CV1 Test at LAST	CV2 Test at 30 °C (54 °F) below LAST	CV2Z, CV2X and CV2ZX Test at 30 °C (54 °F) below LAST
I	220-275 (32-40)	20 J (15 ft-lbs)	no test	X	X	Not applicable
II	> 275-395 (> 40-57)	35 J (25 ft-lbs)	no test	X	X	X
III	> 395-455 (> 57-66)	45 J (35 ft-lbs)	Combination not allowed	X	X	X
IV	> 455-495 (> 66-72)	60 J (45 ft-lbs)	Combination not allowed	X	X	X
V	> 495 (> 72)	60 J (45 ft-lbs)	Combination not allowed	Combination not allowed	X	X
X denotes required tests to minimum Charpy toughness at the specified temperature						

19.5 Applicable steels

Annex C lists commonly used specifications for steel plates, steel shapes (open sections), and steel tubulars, within the context of the MC approach.

Annex D lists commonly used specifications for steel plates, steel shapes (open sections), and steel tubulars, within the context of the DC approach.

Steels that are not included in either list may be used, as long as they satisfy the provisions of Clauses 19 and 20.

19.6 Cement grout for pile-to-sleeve connections and grouted repairs.

19.6.1 Grout materials

High sulphate-resisting portland cement or API standard oilwell cement grouts, mixed with fresh water should be used. Sea water shall not be used in cement and grout mixes due to chemical attacks, potential corrosion and other potential adverse durability effects.

Water should be freshly drawn and be free of hydrocarbons and other deleterious matter.

In grout mix composition, the water/cement ratio shall not exceed 0,40.

Grout mix design should be submitted for approval by the owner prior to works.

High alumina cement (HAC) grout shall only be used with extreme care, and restricted to below splash zone applications in order to reduce conversion and associated strength loss.

Admixtures may be used to improve the grout slurry properties, provided it can be demonstrated that they do not adversely affect the required grout durability and performance.

Grout constituent materials should be supplied to a recognized national standard and stored in accordance with the manufacturer's recommendations.

19.6.2 Onshore grout trial

In the absence of previous production data, the compressive strength of the grout mix shall be confirmed by an onshore trial. An onshore trial shall consist of at least three separate batches of grout. A minimum of six cubes shall be cast from each batch for each age at which strength is to be determined. The compressive strength of 75 mm cubes at 28 days should be used as the basis for strength compliance. However, it is recommended that strength gain with age should be measured at 30 hours, at 3 days and at 7 days, or at other ages as considered appropriate. Cubes of other sizes may be used provided appropriate conversion factors relating the strength to a 75 mm cube are available. Cylinders shall not be used.

The mean cube strength calculated from the data should be greater than either

$$f_{\mathrm{cu}} + 1{,}64\sigma\left(0{,}86 + \sqrt{2/n}\right) \qquad \text{for } 10 \leqslant n \leqslant 100 \qquad (19.6\text{-}1)$$

or

$$f_{\mathrm{cu}} + 1{,}64\sigma \qquad \text{for } n \geqslant 100 \qquad (19.6\text{-}2)$$

where

f_{cu} is the required specified minimum compressive strength;

σ is the standard deviation of the data;

n is the number of cubes.

The grout mixing equipment and mixing times used in the onshore trials, or in previous production grouting from which data is taken, should both be the same as those to be used during offshore grouting to ensure the grout slurry is similarly sheared.

The specific gravity of the grout slurry should be correlated against the mean compressive strength for offshore monitoring.

19.6.3 Offshore grout trial

Prior to grouting, an offshore trial shall be performed, using the equipment and materials to be used for the production grouting, to check the 30 hour grout strength compliance. Grouting shall not proceed until compliance has been achieved. Compliance is satisfied if all cube results exceed the specified minimum 30 hour compressive strength, calculated as defined in 19.6.2.

19.6.4 Offshore quality control

Specific gravity measurements of the grout slurry prior to pumping shall, as a minimum, be recorded once per batch or once per cubic metre, whichever is the more frequent, and preferably should be measured and recorded continuously. The specific gravity of the in-place grout slurry should also be measured at the most appropriate of the following locations: surface returns, the highest point within the grout volume, or from a suitably sized vertical standpipe. An in-place specific gravity greater than or equal to that determined during the onshore trial shall be achieved. Specific gravity measurements shall be made using a pressurized fluid density balance in accordance with ISO 10414-1 or monitoring devices calibrated to this type of balance.

In situations where subsea measuring devices fail, a percentage over volume may be pumped in lieu of direct measurements, if the following conditions are met:

a) it can be demonstrated that on similar connections on the structure, where measurements are available, the required specific gravity was achieved with a recorded net level of grout over volume having been pumped;

b) an additional 20 % in-place volume of grout over and above the net recorded volume in a) above is pumped;

c) no major grout system losses are evident.

During the grouting operation, samples shall be taken from randomly selected batches. At least four samples, each of three cubes, should be taken for each grout volume to give a representative distribution throughout the in-place grout. The cubes should be cast and tested to a recognized standard and match cured to the in-place grout environmental conditions. At least one cube should be tested from each sample to confirm compliance at 28 days. Compliance is satisfied if all individual cube results, in a set of four, exceed the specified minimum compressive strength, f_{cu}, as defined in 19.6.2.

20 Welding, fabrication and weld inspection

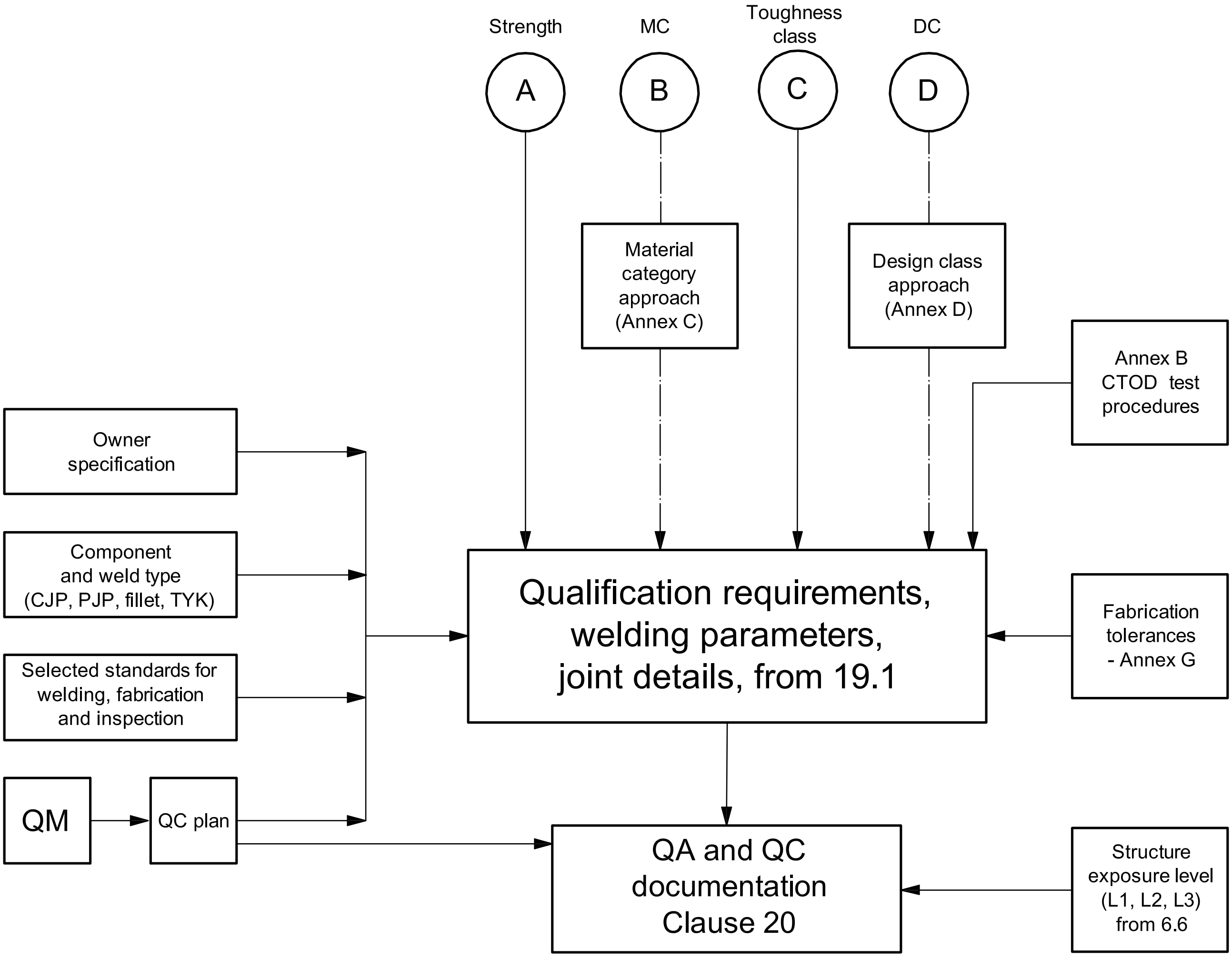
20.1 General

This clause is to be used in conjunction with owner specifications, selected international, national or regulatory standards for welding, fabrication and inspection. It describes complementary provisions for welding, inspection, fabrication, and quality issues, in which requirements specific to fixed steel offshore structures are adapted to steel strength, toughness class and criticality of the structure or component.

An overview of the process is given in Figure 20.1-1. Inputs to procurement and construction are derived from engineering design processes described in Clause 19 and Annexes C and D.

The provisions of Clause 20 shall be supplemented by Annexes B and G, as well as by either Annex E or F, as appropriate.

Guided by the constructor’s embedded quality management system (QMS) a project-specific quality control plan provides proactive QC inspections and controls (e.g. qualification) prior to and during fabrication (see Clause 21).



Key

CJP complete joint penetration

PJP partial joint penetration

TYK T-, Y- or K-Joint

QM quality management

Figure 20.1-1 — Flow chart for welding, inspection, fabrication and QA/QC Documentation

20.2 Welding

20.2.1 Selected generic welding and fabrication standards

The welding of fixed offshore structures shall be performed in accordance with selected generic international or national standards that complement requirements given herein to jointly address, at a minimum, the following topics:

a) applicable welding processes;

b) approved consumables (electrodes, flux, etc.);

c) approved base metals;

d) interchangeability of steels and consumables within groups;

e) matching, undermatching and overmatching of base metals and consumables;

f) minimum/maximum preheat and interpass temperatures;

g) welding procedure and welding personnel qualification, including

   1) weld metal tensile testing and CVN testing of weld metal and heat affected zone (HAZ) (for procedures only),

   2) range of applicability (positions, thicknesses, etc.),

   3) essential variables,

   4) number and type of test specimens,

   5) test specimen descriptions,

   6) test specimen locations,

   7) test specimen acceptance criteria, and

   8) welding of single sided T-, Y- and K-joints;

s) production weld joint details and tolerances, including

   9) welding of single-sided T-, Y- and K-joints, and

   10) other welds;

t) post-weld heat treatment;

u) heat straightening limits;

v) weld size/profile limits;

w) environmental limitations for welding;

x) back-gouging/backing/runoff tab criteria;

y) preparation of base metal (cleaning, bevelling, etc.);

z) fatigue improvement techniques (if applicable);

aa) NDT procedures;

bb) NDT discontinuity acceptance criteria;

cc) repairs.

The standard or standards used to address a) to r) above can be generic documents applicable to a wide range of steel structures. In the remainder of this clause, these standards are referred to as “the selected standard”. The requirements of this clause deal with issues specific to fixed offshore structures and shall be used to supplement the selected standard.

For a particular project application, the owner shall identify suitable standard(s) to satisfy the above requirements. Examples are given in A.20.2.1.

20.2.2 Weld metal and HAZ properties

20.2.2.1 General

Subclause 20.2.2 describes required properties for weld metal and heat affected zones for the strength grade and toughness class of the connected base metal.

Strength matching shall be as specified in the selected welding standard (20.2.1). Where undermatching is permitted, this shall be as specified in the design. Where materials of two different strengths are joined, the protocol for matching shall be as described in design documents.

20.2.2.2 Material category (MC) toughness

Annex E defines the minimum toughness requirements for weld metal and HAZ, respectively, for use with the MC approach. Group V steels are not fully described, as these steels require the designer to specify the minimum toughness requirements.

20.2.2.3 Design class (DC) toughness

Annex F defines the minimum toughness requirements for weld metal and HAZ, for use in the DC methodology. Groups IV and V are not described, as these steels require the designer to specify the minimum toughness requirements.

20.2.2.4 Charpy V-notch (CVN) toughness

20.2.2.4.1 Testing

For specimen tests, the selected welding standard’s requirements shall be followed.

The test temperatures and minimum energy values in weld metal and HAZ shall be as required to match the performance of the given steels to be joined as tabulated in Annex E or F.

20.2.2.4.2 Additional essential variables

For welding procedures that are Charpy-tested, the following changes shall be considered essential variables, in addition to the essential variables listed in the selected welding standard.

a) An increase in carbon equivalent value, P_{CE}, computed from Equation (20.1-1), greater than 0,03 %, or an increase in modified carbon equivalent parameter for cracks, P_{CM}, computed from Equation (20.1-2), greater than 0,02 %, with respect to the values specified in the procedure qualification test:

$$P_{CE} = P_{C} + P_{Mn}/6 + (P_{Cr} + P_{Mo} + P_{V})/5 + (P_{Ni} + P_{Cu})/15 \qquad (20.1\text{-}1)$$

$$P_{CM} = P_{C} + P_{Si}/30 + (P_{Mn} + P_{Cu} + P_{Cr})/20 + P_{Ni}/60 + P_{Mo}/15 + 5\,P_{B} \qquad (20.1\text{-}2)$$

where

P_{C} is the weight percentage of carbon;

P_{Mn} is the weight percentage of manganese;

P_{Cr} is the weight percentage of chromium;

P_{Mo} is the weight percentage of molybdenum;

P_{V} is the weight percentage of vanadium;

P_{Ni} is the weight percentage of nickel;

P_{Cu} is the weight percentage of copper;

P_{Si} is the weight percentage of silicon;

P_B is the weight percentage of boron.

dd) An increase of more than 28 °C (50 °F) over the maximum interpass temperature attained during procedure qualification test.

ee) A change greater than 25 % in the heat input of the welding passes from the average value calculated in the supporting procedure qualification test. Heat input average for fill passes shall be calculated separate from cap and root passes.

ff) Any change in electrode or electrode/flux brand name or place of manufacture when the CVN test temperature is lower than – 18 °C (0 °F).

20.2.2.5 CTOD toughness

20.2.2.5.1 General

CTOD tests are only required for CV2X and CV2ZX steels. For testing details, see Annex B.

CTOD testing of the HAZ may be omitted if the requirements of 20.2.2.5.2 are met. Heat-affected zone CVN test results on the procedure qualification record (PQR) should be greater than or equal to the results obtained by the steel manufacturer during pre-production qualification.

20.2.2.5.2 Pre-production qualification

CTOD tests of the HAZ may be omitted if the steel is pre-production qualified by the manufacturer in accordance with a recognized standard.

CTOD tests shall be performed for each condition of material supply from each manufacturer. Essential variables to be considered in material supply shall include melting practice, rolling practice, maximum thickness, specific chemical composition range and plate manufacturing process (e.g. normalized, TMCP or Q&T).

Repetitions of HAZ CTOD testing may be waived for tests in which the CTOD properties are not expected to be degraded (e.g. where a second procedure is within the range of heat input and preheat/interpass temperatures of those to be qualified by a first).

CTOD data for the HAZ is generally provided by the steel supplier’s weldability data. However, specific CTOD testing of the HAZ of the test weld may be specified in the welding procedure qualification requirements. When reliance is placed on the steel supplier's CTOD tests, Charpy test results on the PQR should not be substantially lower than those obtained by the steel supplier in parallel with the CTOD test.

Qualification welding shall be conducted at both the highest heat input/interpass temperature and lowest input/interpass temperature to be allowed for production welding, as governed by the welding procedure specification (WPS).

The test welding positions shall be selected as those representing the highest and lowest heat input of the procedure(s) to be qualified by such tests.

20.2.2.5.3 CTOD fracture toughness requirements

CTOD requirements are a function of design philosophy. The test temperature shall not be greater than the minimum design temperature for a given service.

All tests shall meet the target CTOD acceptance level set by the designer.

Representative CTOD requirements range from 0,10 mm at 4 °C (0,004 in at 40 °F) to 0,38 mm at – 10 °C (0,015 in at 14 °F). Achieving the higher levels of toughness can require some difficult compromises, set against other desirable attributes of the welding process — for example, the deep penetration and relative freedom from trapped slag of uphill passes.

CTOD testing reports complying with the testing standard shall be included in the PQR documentation. Photographs of at least 1× magnification, clearly showing the fatigue pre-crack and the fracture face of one of the broken halves of all CTOD specimens, shall be included in the PQR documentation.

20.2.2.5.4 Additional essential variables

For welding procedures that are CTOD-tested, the following changes shall be considered essential variables, in addition to the essential variables listed in the selected welding standard:

a) any increase in the maximum interpass temperature over that qualified in the high heat input CTOD procedure qualification test;

b) an increase in joint thickness greater than that used for the CTOD procedure qualification tests;

c) a decrease in heat input from the heat input qualified during low heat input CTOD procedure pre-qualification test for the HAZ;

d) any increase in the heat input from the average heat input qualified of the fill passes during the high heat input CTOD procedure qualification tests;

e) any change in electrode or electrode/flux brand name or the place of manufacture;

f) a reduction in the depth of back-gouging as measured during the CTOD qualification testing;

g) a reduction in the width of back-gouging as measured during the CTOD qualification testing;

h) an increase in the layer thickness (as measured at the fusion line) greater than 10 %, versus that measured during CTOD pre-qualification testing.

20.2.2.5.5 Qualification range

The CTOD qualification tests qualify production welding thickness up to and including the thickness tested.

20.2.2.5.6 Documentation and results

CTOD testing reports shall be included in the PQR documentation. Photographs of at least 1× magnification, clearly showing the fatigue pre-crack and the fracture face of one of the broken halves of all CTOD specimens shall also be included.

20.2.2.6 Hardness testing

A Vickers hardness survey shall be performed on one of the macro sections from each test weld in accordance with a recognized standard, using an applied load of 100 N or less. The sample shall be ground, polished, and etched with a suitable solution to give clear definition of the weld, heat affected zone, and fusion line microstructures under the microscope.

Hardness test locations shall be located as detailed below and as shown in Figure 20.2-1:

a) HAZ — starting at 0,25 mm to 0,50 mm from the fusion line with three indentations at 0,50 mm to 0,75 mm intervals;

b) weld metal — a minimum of three equally spaced indentations for each hardness traverse.

The hardness traverse on fillet welds shall be similar to that for T-connections.

Maximum hardness values shall not exceed 325 HV10 for below water locations and 350 HV10 for above water. If any individual hardness value exceeds the limit specified, two further sections shall be taken, both of which shall meet the specified requirements.

The maximum HAZ hardness limit of 325 HV10 is considered an acceptable safe level for typical carbon and low alloy structural steels. Weldability tests have shown that critical hardness decreases with decreasing carbon equivalent values (P_{CE} and P_{CM}, see A.19.3.2 and A.19.3.3). Newer low carbon structural steels can crack at maximum HAZ hardness of 325 HV10 and less.

Hardness test sample locations for qualification tests performed on pipe shall be at the 3 o'clock or 9 o'clock position for downhill cap and at the 6 o'clock position for uphill cap passes.

A photograph or photo macrograph of 1× to 3× magnification, clearly showing the hardness indentations, HAZ, and the weld zone, shall be included in the test results. For acceptable qualification, the macro-etch test specimen, when inspected visually, shall also conform to the requirements for visual examination.

20.2.2.7 Other mechanical tests

Other mechanical tests shall be conducted during the welding procedure qualification in accordance with the selected standard's requirements. These include, as applicable

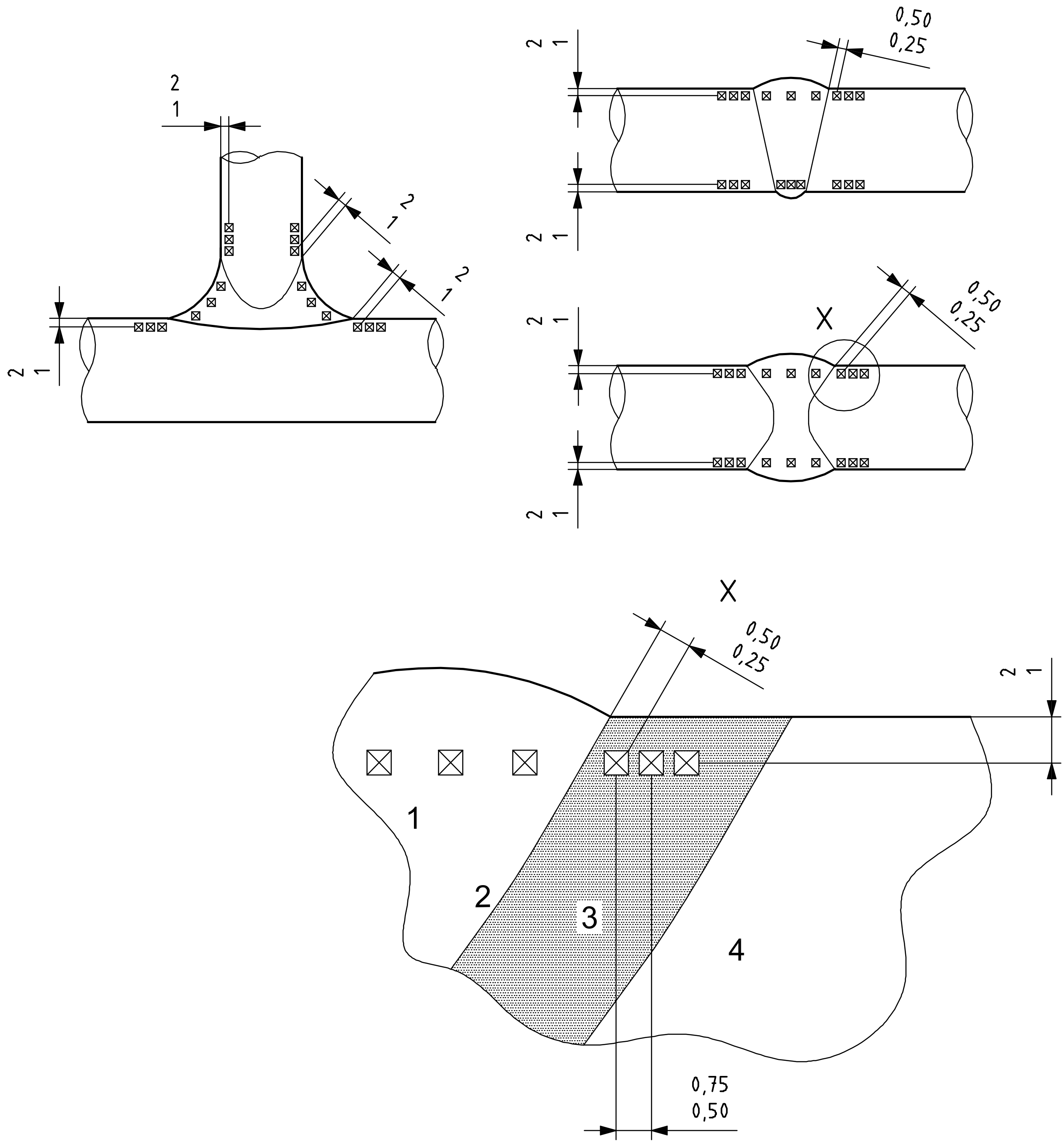
a) tension tests,

b) bend tests, and

c) macro-etch sectioning.

20.2.3 Tubular T-, Y- and K-joints

Welded connections of tubular intersections at T-, Y- and K-joints are a dominant feature of fixed steel offshore platforms. Such welds are made from the outside only in tubular bent (or point to point) fabrication. Here, achieving the performance attributes of complete joint penetration welds requires specialized consideration of geometry and position variations, weld joint bevel preparation, fit-up tolerances, welding procedure qualification, welding personnel qualification, welder qualifications, cap profiling and fitness-for-purpose NDT flaw acceptance criteria. The selected standard should provide such guidance for welding, fabrication, and inspection (see A.20.2).



Key

1 weld metal

2 fusion line

3 HAZ

4 base metal

X test location

Figure 20.2-1 — Vickers hardness test locations

20.3 Inspection

The inspection requirements for the MC and DC methodology are described in Annexes E and F, respectively.

Additional requirements for QC, QA and documentation are presented in Clause 21.

20.4 Fabrication

20.4.1 General

The following complementary requirements pertain specifically to fixed steel offshore structures and shall take precedence over selected standards for generic industrial applications.

20.4.2 Weld requirements

20.4.2.1 Splice welds

Splices shall be fabricated using full penetration welds.

20.4.2.2 Weld proximity

Weld proximity shall satisfy the following requirements.

a) Circumferential welds are not permitted within cones and node stubs unless specified on design or approved shop drawings.

b) The minimum spacing between circumferential welds in tubular members shall be the smaller of 915 mm (36 in) or the diameter of the member.

c) Ring stiffeners in tubular members shall be at least 100 mm (4 in) from circumferential welds, except that, where this is not possible, the welds shall overlap by at least 10 mm (3/8 in).

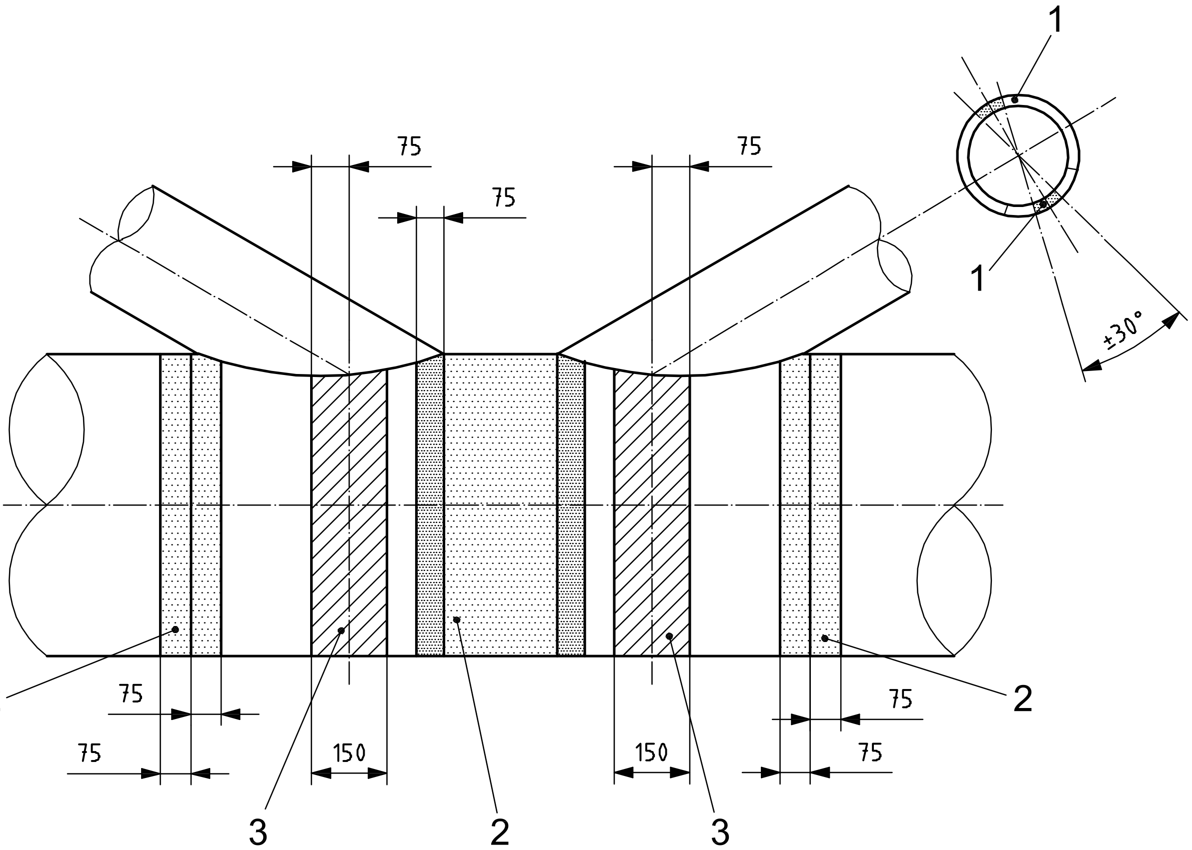
d) Rings used to stiffen cone-cylinder junctions shall not be offset and the welds shall be overlapped by at least 10 mm (3/8 in), to avoid coincident location of weld toes.

e) Longitudinal and circumferential welds in tubular joints shall avoid the most critical areas of a joint, as shown in Figure 20.4-1.

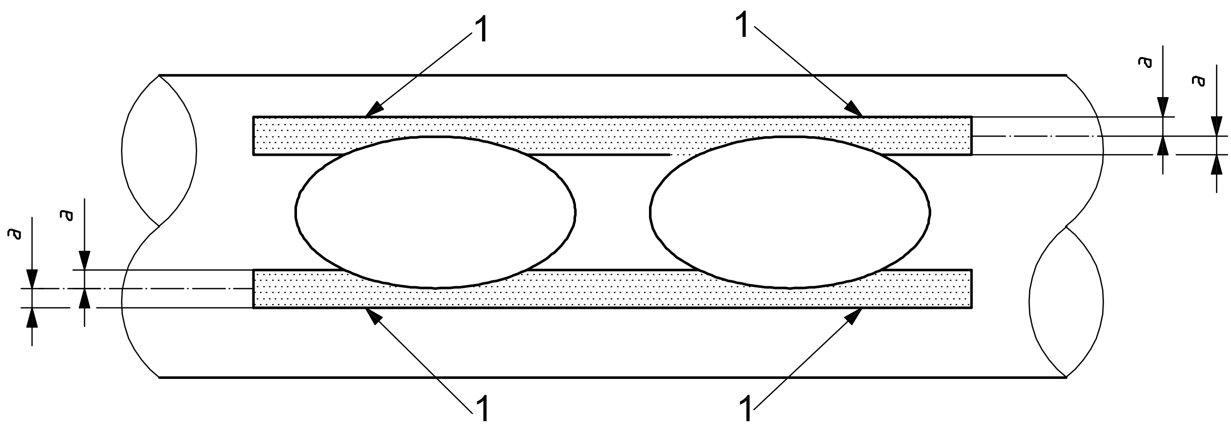
f) The offset between longitudinal seams in adjacent cans in tubular joints shall be maximized, taking into account connections at each end of each member, and shall be in no case less than 90°, except that the location of a longitudinal seam may be adjusted by no more than 300 mm (12 in) to avoid the prohibited locations as shown in Figure 20.4-1 b).

g) In any members to be slotted to receive gusset plates, the slot shall be at a distance of 300 mm (12 in) or twelve times the member wall thickness, whichever is greater, from any circumferential weld. To avoid notches, the slotted member shall be drilled or cut and ground smooth at the end of the slot with a diameter of at least 3 mm (1/8 in) greater than the width of the slot. Where the gusset plate passes through the slot, the edge of the gusset plate shall approximately conform to the shape of the slot to provide a better fit-up and welding condition.

Dimensions in millimetres



a) Prohibited locations for circumferential welds on chords and longitudinal seams on stubs



b) Prohibited locations for longitudinal seams on chords

Key

1 longitudinal seams not permitted in these areas

2 circumferential welds not permitted in these areas

3 circumferential welds should be avoided in these areas

[a] Lesser of 150 mm or twice the chord thickness (measured along the chord circumference).

Figure 20.4-1— Positioning of longitudinal and circumferential welds in tubular joints

20.4.2.3 Fabrication aids

When fabrication aids and temporary attachments occur in areas that become part of a completed weldment within 300 mm (12 in) of an intersecting weld or in areas that require coating, the attachment shall be removed by oxy-fuel, cutting a minimum of 3 mm (1/8 in) above the surface of the structural member and grinding the remainder flush with the surface of the member. If such ground areas lie close to or underneath a weld, the surface shall be inspected using MPI after grinding and before the permanent weld is executed.

Fabrication aids and temporary attachments that occur in areas that do not require coating and that are located more than 300 mm (12 in) from an intersecting weld shall be seal welded and shall be removed to within 6 mm (1/4 in) of the surface of the member with sharp edges removed by grinding. Oxy-fuel cutting may be used for the bulk removal of a fabrication aid or temporary attachment, but shall not be used within 3 mm (1/8 in) of the surface of the underlying component. Fabrication aids and temporary attachments shall not be removed by hammering or oxy-fuel washing.

20.4.3 Forming

Guidelines for forming steel plates into a circular tubular member are given below.

For a diameter-to-thickness (D/t) ratio of less than 20, mechanical tests shall be undertaken to demonstrate that the steel retains its mechanical properties. Post-forming heat treatments, or hot forming, may be used to mitigate the effects of cold forming.

Any materials that require straightening before use to achieve the specified tolerances may be cold straightened if the strain does not exceed 5 %. Similarly, tubular cans for members or joints may be re-rolled after welding to meet the dimensional tolerances if the strain during re-rolling does not exceed 5 %.

Consideration should be given to strain concentration and discontinuity growth in the weld metal or HAZ, if the nominal strain during re-rolling exceeds 4 %, or the product of SMYS times thickness exceeds 3 200 MPa · cm (180 ksi · in).

Re-rolling, hot forming, or post-forming heat treatments, if used, shall be represented in welding procedure qualification (WPQ) testing of weld metal and HAZ.

20.4.4 Fabrication tolerances

The location of each member of the structure shall be in accordance with the design drawings within the final fabrication tolerances. The fabrication tolerances are given in Annex G. Tolerances not stated in Annex G shall follow applicable national standards.

Tolerances should be checked at each stage in accordance with the fabrication procedures and the final survey shall meet the defined tolerances. Fitters should perform their own checking as the job progresses; their tape measures and straight edges do not need calibration. The final survey shall be on complete sub-assemblies and on the completed structure after PWHT, if applicable.

The personnel responsible for the final survey should be either qualified surveyors or have had at least five years experience of similar work. Instruments used shall be in accurate adjustment and shall have current valid calibration certificates.

Allowance shall be made for weld gap tolerances and weld shrinkage in all components, sub-assembly and global tolerance calculations.

Where tolerances have to be derived from a formula (i.e. tolerances expressed in terms of the component dimension; e.g. wall thickness), the results shall be taken to the nearest mm (0,04 in).

20.4.5 Grouted pile-to-sleeve connections

Requirements for the detailing of pile-to-sleeve connections are given in 15.1.3.

Steel surfaces in contact with the in-place grout and on which interface shear is to be developed shall be free of loose material, mill scale, grease and any other material that can reduce the grout to steel friction.

Care shall be taken during and after the installation of packers to prevent damage from handling, high temperatures and splatter from welding and debris. Any debris that could cause damage to the packers during the installation process shall be removed. Measures such as dressing of the pile tip to a smooth profile and positioning of centralisers on the sleeve directly above the packers should be taken to minimize potential packer damage during pile stabbing.

21 Quality control, quality assurance and documentation

21.1 General

This clause provides the quality control (QC), quality assurance (QA) and documentation requirements for fabrication, transportation and installation phases of a typical offshore structure construction project.

QC inspection and testing shall be performed to ensure adherence to the plans and specifications containing the detailed instructions necessary to obtain the desired quality and service in the finished product, during all phases of construction, including the fabrication, loadout, sea-fastening, transportation and installation.

QA/QC, inspection and documentation requirements should be commensurate with the structure’s exposure level (L1, L2, or L3), as described in 6.6, and shall be consistent with welding and fabrication provisions, in accordance with recognized international, *de facto* international, regional or national standards. Additional requirements and modifications to suit the project application can be specified by the owner. These should be coordinated with the material selection and fracture control philosophy used in design.

The extent of non-destructive testing of welds during fabrication shall be in compliance with the inspection category. The selection of an inspection category for each weld should be in accordance with Annex E or Annex F, as appropriate.

The most effective QC scheme is one that prevents the introduction of defective materials or workmanship into a structure, rather than depending upon QA and documentation which come after the fact.

21.2 Quality management system

All design, fabrication, and installation shall be performed under a documented quality management system (QMS), certified to ISO 9000, and following ISO 9001 and ISO 9004, or an equivalent QMS.

The QMS shall, as a minimum and where applicable, address the items listed in Table 21.2-1, in accordance with the structure's exposure level. Items may be covered by reference to the QC plan. The QMS shall relate to the elements of the structure and any additional elements that are critical to the fabrication of the structure, e.g. design and testing of lifting beams and lifting procedures.

Table 21.2-1 — Quality system requirements

Documentation description	Exposure level		
	L1	L2	L3
Quality assurance manual	X	X	—
Organizational chart	X	—	—
Documentation and drawing control procedures	X	X	—
Design change procedures	X	X	—
Design calculations	X	X	—
Fabrication procedures	X	X	—
Lifting procedures	X	X	—
Material tracing control procedures	X	X	—
Dimensional control/survey procedures	X	X	X
Welding consumable storage and handling procedures	X	X	X
Pressure/leak, and equipment operability test procedures	X	—	—
Preheat procedures	X	X	X
NDT procedures	X	X	—
Non-conformance/remedial procedures	X	X	—
Pressure/leak test procedures	X	X	—
Inspection equipment calibration procedures and certificates	X	—	—
Subcontractor quality procedures	X	X	—
Weight reports procedures	X	—	—
Procurement procedures for materials/services	X	X	X
Procedures for handling special processes (e.g. heat treatment)	X	—	—
X minimum requirements for applicable phases of the overall project — not required			

21.3 Quality control plan

21.3.1 General

QC is normally performed as appropriate prior to and during fabrication, loadout, transportation and installation. The purpose of quality control is to ensure that materials and workmanship meet the specified requirements. QA is generally performed during and after QC activities, to provide records and documents.

A QC plan shall be developed which makes reference to procedures for all inspection and NDT techniques, including proposed report formats and the names and qualifications of inspection and NDT personnel.

The QC plan shall define the scope of inspection required for fabrication while meeting the requirements of Clause 20 and either Annex E or Annex F, as appropriate.

The QC plan shall consider the elements in the following subclauses.

21.3.2 Inspector qualifications

Inspectors shall be qualified to carry out their duties by education, experience, or practical testing according to recognized international, *de facto* international, regional, or national, standards.

Personnel who assist inspectors during any phase of construction of an offshore structure should demonstrate ability and experience, or be qualified to an appropriate standard for the required inspection of a particular project.

21.3.3 NDT personnel qualifications

NDT personnel shall be certified to an approved programme consistent with a recognized standard that requires experience and passing written and practical tests applicable to the project inspection methods and type of construction.

21.3.4 Inspection of materials

Inspection shall verify that all materials being incorporated into any portion of the fabrication are in accordance with the specified requirements.

Steel that does not have supporting material traceability record (MTR) documentation from the steel manufacturer or supplier shall not be used.

21.3.5 Inspection of fabrication

Inspections of the structure shall be made upon completion of fabrication to confirm compliance with the dimensional requirements, using the tolerances established by the selected standard and the specific requirements of Clause 20. Inspections made during fabrication shall be used to determine compliance with in-process requirements, such as joint fit-up, that cannot be confirmed upon general fabrication completion.

21.3.6 Inspection of welding

Welding inspection and testing shall be performed to verify compliance with the requirements specified in Annex E or Annex F.

21.4 Inspection of installation aids and appurtenances

Inspections shall verify that all installation aids and appurtenances have been installed and tested in accordance with the specified requirements, including any manufacturer's recommendations.

Installation aids include the following:

- launch systems;
- flooding systems;
- grouting systems;
- mudmats;
- jetting systems;
- lugs and guides;
- monitoring systems;

Appurtenances include the following:

- boat landings;
- riser guards;
- risers and clamps;
- J tubes;
- sump and pump caissons.

During transportation and installation a structure can carry piles and conductors. The inspections shall also verify that these are installed and tested in accordance with the specified requirements.

The location, size, and orientation shall be checked, and weld attachments (including temporary restraints) shall be subjected to NDT in accordance with Clause 20.

Inspections shall include functional tests of all mechanical and electrical equipment and systems, including instrumentation. Cabling and instrumentation shall be checked to ensure continuity, and all hydraulic and pneumatic lines shall be pressure tested. All non-steel components (i.e. diaphragms, packers, valve seats, etc.) shall be protected from damage from weld spatter, debris, and/or any other construction activities, and hydraulic lines shall be thoroughly flushed and drained before and after testing. The inside of legs, skirt piles, etc. shall be inspected to ensure complete removal of debris (e.g. welding rods, miscellaneous pieces of wood, steel, etc.), which could damage non-steel components during installation.

21.5 Inspection of loadout, sea-fastening and transportation

Inspection shall be performed for all areas related to loadout, sea-fastening and transportation, to confirm compliance with the specified requirements. Prior to loadout, a final visual inspection of the structure shall be conducted to ensure that all components are in place, all welds have been properly completed and inspected, all temporary transportation/installation aids are included and secure, all hydraulic and pneumatic lines have been properly installed, tested, flushed and secured, all temporary fabrication aids and debris have been removed, and that all temporary welded attachments have been removed and attachment marks repaired, according to the specified requirements.

The support foundations, including the loadout pathway, the dock, the transport vessel and the sea floor at the dockside shall be inspected, in order to ensure compliance with specified requirements.

Other areas for inspection include the lifting/pulling/pushing components attached to the structure (which require NDT) and those between the structure and lifting equipment (i.e. lifting slings, shackles, spreader beams). For vendor supplied items, documentation is required in addition to the inspections. The capacity and condition of loadout equipment shall be confirmed by inspection and documentation.

For skidded loadout, inspection shall be performed to confirm that the skidway and/or launch surface is clean and properly lubricated (if required) prior to loadout. The winches, jacks, and pulling cables shall be inspected for proper capacity and condition.

If ballast and deballast operations are required to compensate for tidal variations, inspection of the ballast system is required to confirm adequacy and condition of the equipment. The operation should be monitored to ensure compliance with the loadout procedure.

Inspection for sea-fastening of the structure and all deck cargo is required to confirm compliance with the specified requirements. This includes temporary tie-downs and bracing required for transport. Materials, fabrication, and weld inspection requirements shall be in accordance with Clauses 19 and 20. Inspection for structure launch items shall be conducted where possible prior to sea transport.

Seaworthiness of tugs, towing attachments and the transport vessel shall also be confirmed. For preparation of self-floating structures for transport to location, visual inspection shall be performed to confirm seaworthiness and that all towing/restraining lines are properly attached.

21.6 Installation inspection

21.6.1 Structure launch and upending

Prior to launch, visual inspection shall confirm that all tie-downs and temporary bracings are cut loose, and tow lines and loose items are removed from the launch barge or safely secured. Inspection is required to confirm that the structure flooding system is undamaged, flooding valves are closed and that the launching arm system is in the proper mode of operation. For lifted structures, inspection shall confirm removal of all restraints and proper attachment of lifting equipment, as well as the undamaged and properly configured operation mode of the flooding system. For self-floating structures, inspection shall confirm removal of tow lines, as well as the undamaged and properly configured operation mode of the flooding system.

Inspection should be carried out after the structure is secured in place. If inspection is necessary before then (i.e. inspection for suspected damage to the flooding system), it should be limited to those items required to upend and secure the structure.

21.6.2 Piling and conductor installation

All pile and conductor welds performed during fabrication shall be inspected in accordance with the requirements of Clause 20 prior to loadout, including lifting devices, lugs and attachments. During installation, inspection shall be conducted to ensure that the correct pile make-up is followed and that the welding of add-on sections (if applicable) is performed in accordance with the specified requirements.

Prior to each use, pile hammers shall be inspected for proper hook-up and alignment for operation.

During pile installation, NDT shall be performed

a) on the welded connections at pile add-ons,

b) between pile and deck support members, and

c) between the pile and leg,

as well as elsewhere, to confirm compliance with the specified requirements. NDT inspection shall be performed in accordance with either Annex E or Annex F, as appropriate.

21.6.3 Topsides installation

Prior to lifting, visual inspection shall be performed to confirm that tie-downs and other items not considered in the lifting design are removed from the topsides. Proper rigging and connection of all lifting components shall also be confirmed.

Immediately after lifting, visual inspection shall be performed on all scaffolding and other temporary support systems to confirm their adequacy for completion of weld out. Materials, fabrication and welding requirements shall be in accordance with Clauses 19 and 20. Visual inspection shall be performed on the structure and deck mating points to confirm proper alignment and fit-up and to ensure that weld preparations are as per specified requirements. Following weld-out, inspection shall be performed on the welded connection in accordance with Clause 20 and/or other specified requirements.

These inspections shall be performed for each component of a multiple lift, with inspection for alignment during each lift.

21.6.4 Underwater inspection

In the event the installation requires underwater operations, the inspection shall verify, either by direct communications with divers or through the use of a remote monitoring device, that the operation has been conducted in accordance with the specified requirements.

21.7 Documentation

21.7.1 General

During the structure's fabrication, construction, loadout, and installation, inspection-related data are generated. This information, along with any pertinent additional data shall be recorded as the job progresses, and shall be compiled in a timely manner and in a form suitable to be retained as a permanent record.

The inspection results and other documents listed in Table 21.7-1 shall be prepared. All documentation referenced in 21.7 shall be retained on file for the life of the structure.

Table 21.7-1 — Documentation requirements

Documentation description	Exposure level		
	L1	L2	L3
Material tracing records	X	X	—
Engineering drawings	X	X	X
Shop drawings	X	X	—
Design calculations for construction activities	X	X	
Dimensional control/survey records	X	—	—
Mill certificates	X	X	—
Welding procedure matrix	X	X	—
Welding procedure specification	X	X	X
Welding procedure (pqr) test results	X	X	X
Site welding instruction sheets (short wps)	X	X	—
Welder qualifications	X	X	X
NDT test results	X	X	—
NDT personnel qualifications	X	X	—
Inspection records	X	X	—
Pressure/leak and equipment operability test reports	X	X	—
Weight reports	X	—	—
Other inspection	X		
Foundation verification report (pile driving records, etc.)	X	X	X
As built drawings	X	X	—
Weld numbering system	X	X	—
X minimum requirements for fabrication — not required			

21.7.2 Calculations

The structural integrity of components and adequacy of equipment during all phases of construction shall be addressed.

Calculations should

a) address the structure, attachments, temporary works, cranes, and rigging,

b) address structural strength and stability,

c) address all actions, stresses, deflections, equipment, and rigging, and

d) reference appropriate drawings and specifications.

Calculation documentation should provide the source of calculation methods.

21.7.3 Weight and centre of gravity reports

Weight and centre of gravity reports shall be based on design, shop or as-built drawings as the job progresses.

NOTE Guidance on weight control is given in ISO 19901-5[5].

21.7.4 Fabrication inspection documentation

Documentation related to fabrication activities shall be provided in accordance with Table 21.7-1, as appropriate.

21.7.5 Installation inspection documentation

Documentation for materials, testing and welding inspection performed during the installation phase shall be recorded and retained. Pile blow count with depth and final pile penetration shall be documented, and a continuous log of events, including climatic conditions (i.e. temperature, wind, barometric pressure, humidity), sea states, operational activities, etc. should be retained.

21.8 Drawings and specifications

The drawings and specifications prepared through the course of the project shall comprise the following.

a) **Conceptual drawings** (see A.21.8.2)

gg) **Bid drawings and specifications** (see A.21.8.3)

hh) **Design drawings and specifications** (see A.21.8.4)

ii) **Fabrication drawings and specifications** (see A.21.8.5)

The contractor shall prepare and furnish fabrication procedures and assembly drawings describing and showing the proposed methods and order of assembly on the structure. These procedures and drawings shall include rigging components, rigging configuration, lifting crane capacity and location, temporary aids and attachments to the structure.

jj) **Shop drawings** (see A.21.8.6)

Shop drawings shall include all shop details, including material types, cuts, copes, joint details, holes, bolts and piece numbers in accordance with the contract drawings. All welds shall be identified according to Clause 21.

kk) **Installation drawings and specifications** (see A.21.8.7)

ll) **As-built drawings and specifications** (see A.21.8.8)

The contractor shall prepare and furnish as-built versions of the design drawings that accurately reflect the structure as built. These will be clear and legible, permanently reproducible drawings with all changes highlighted by clouds around the details concerned. Only those dimensions outside the fabrication tolerances shall be marked on the as-built drawings.

22 Loadout, transportation and installation

22.1 General

22.1.1 Planning

The installation of a platform consists of loading out and transporting its various components from the fabrication site to the installation location, positioning the structure on location, and assembling the various components into a stable structure in accordance with the design drawings and specifications.

An installation plan shall be prepared for each platform. This plan should include the methods and procedures developed for the loadout, the sea-fastening, the transportation of all components, the complete installation of the structure, with piles/conductors and topsides, as well as include the equipment used in executing the procedures. The information may be provided in the form of written descriptions, specifications, and/or drawings. Depending upon the complexity of the installation, more detailed instructions can be required for special items such as grouting, diving, welding and inspection. Any restrictions or limitations to operations due to, for example, environmental conditions, barge stability or structural strength (e.g. lifting capacity), should be stated.

The installation plan is normally subdivided into phases, for example, for the structure: loadout, transportation, transfer from the transport barge into the water, placement on the sea floor, pile installation, conductor installation and topsides installation.

Guidance on marine operations is also given in ISO 19901-6[3]; references to Clause 8 should also be taken as references to ISO 19901-6, where applicable.

22.1.2 Records and documentation

During loadout, transportation and installation, all daily reports, logs, test reports, pile driving records, etc. shall be prepared, compiled and retained. These documents should also record any variation from intended installation procedures and all unusual environmental conditions that occurred during the installation. Any field modifications made shall be noted to record the as-built condition of the structure. At the completion of the work, all documentation shall be collected and retained by the owner (see Clause 21), together with other documents related to the construction and inspection of the structure.

22.1.3 Actions and required resistance

The actions associated with each phase of the installation shall be calculated in accordance with Clause 8. Structural analysis and design shall be performed in accordance with the appropriate clauses of this International Standard to ensure that the structure has sufficient strength and stiffness to resist the internal forces from all relevant combinations of actions.

The structural strength and deformation of any installation equipment (barges, tie-downs, etc.) should also be considered in as much as they can affect the structure.

22.1.4 Temporary bracing and rigging

The requirements of this International Standard shall also apply to any installation aids, temporary struts, bracing or rigging required during any phase of the installation. Any welding of installation aids, temporary struts, or bracing to the structure shall be in accordance with Clause 20.

22.2 Loadout and transportation

22.2.1 General

The transfer of all parts of a platform from their fabrication sites to the installation location presents a complex task which requires detailed planning. Detailed considerations vary with the type of platform to be transported, but the following aspects shall be considered where appropriate.

A platform normally consists of a structure, piling, topsides and other miscellaneous items. These parts are usually transported to site as deck cargo on barges or other vessels. In some cases, the structure is designed with sufficient buoyancy to be floated to site without the use of a barge; such buoyancy can include temporary buoyancy removed after installation of the structure.

22.2.2 Loadout

Loadout shall be performed in accordance with the requirements of the installation plan. The plan should include allowable environmental conditions during loadout operations and design environmental conditions for any mooring system. All items of cargo should be positioned on the barge as shown on loadout plans.

If the structure is loaded out onto a free-floating barge, the barge's ballast system shall be capable of compensating the changes in draught, trim, heel and tide. An adequate standby ballast system should be provided.

If the structure is loaded out onto a grounded barge, it shall be demonstrated by analysis or by previous experience that the barge has sufficient structural strength to distribute the concentrated actions on its deck to the supporting foundation. In addition, the seabed or pad should be smooth, level and free of any obstructions that could damage the hull.

Actions resulting from the loadout operation are addressed in 8.5.

22.2.3 Cargo and launch barges

22.2.3.1 General

An adequate number of seaworthy cargo barges shall be provided. The barges selected shall be of proper size and structural strength to ensure that the stresses in the barge, cargo and sea-fastening during loadout and transportation are within acceptable limits. If the structure is to be launched from a barge without the use of a crane barge, the launch barge shall be suitable for this operation.

22.2.3.2 Barge strength and stability

The structure and any other platform part to be transported on the same barge shall be loaded out in such a manner that a balanced and stable condition is ensured at all times. The barge trim is then adjusted, and the barge is ballasted to the appropriate draught for transportation to site. Such operations should be performed at the dockside or in a sheltered area before reaching open water and before sea-fastenings are attached.

Static and dynamic stresses in the barge hull and framing due to loadout (e.g. from localized actions), transportation and launching should be in accordance with the requirements of the classification society responsible for classification of the barge.

The stability of the barge together with its cargo shall be verified to comply with international regulations and with the flag state requirements.

22.2.3.3 Sea-fastenings

Adequate sea-fastenings shall be designed and installed for all cargo components to prevent shifting while in transit. The sea-fastenings shall be described and detailed in the installation plan. They shall be attached to the structure, topsides and other platform components only at locations approved by the structure designer. Additionally, they shall be attached to the barge at locations capable of distributing the actions to the internal framing. The sea-fastenings should be designed to facilitate their easy removal on-site.

Sea-fastenings shall be designed for actions calculated in accordance with 8.6. Where there is substantial operational experience for routine installations, for similar cargoes, in a particular area and at a similar time of year, then sea-fastening arrangements may be based on that previous experience.

In lieu of a specific assessment for the transportation being considered, the sea-fastenings may be designed for environmental conditions determined from the time required to transport the barge to shelter. The probability of exceeding the selected environmental conditions during such time should be in the range of 1 % to 5 %, taking into account the season of the year in which the tow will take place and the length and reliability of the weather forecast.

22.2.4 Towing vessels

A sufficient number of seagoing tugs shall be provided, with ample power and size to operate safely for each particular route or ocean to be travelled. The size and power requirement of the towing vessel and the design of the towing arrangement shall be calculated or determined from past experience.

The selection of towing vessels should consider the length of the tow route, the proximity of sheltered water, and the weather conditions and sea states expected for the season of the year. When more than one towing vessel is required, the total calculated required bollard pull should be increased to take into account the loss of efficiency due to a multiple tow.

A standby or alternative towing line should be provided and rigged for easy access, in the event of tow line failure.

22.2.5 Actions on the platform components

Consideration shall be given to the actions applied to the various platform components as they are lifted on and off the barge, or as they are rolled on, and launched from, the barge. The actions, inclusive of the actions from transportation, shall be determined in accordance with Clause 8. The resistance to such actions shall be determined in accordance with Clauses 13 to 16, as appropriate.

22.2.6 Buoyancy and flooding systems

22.2.6.1 General

Any platform part intended to float during transport and/or installation shall be checked to ensure adequate buoyancy for flotation, with sufficient reserve buoyancy for the planned operations. All components shall be designed to withstand the actions associated with transport and/or installation, including hydrodynamic pressures during launching or those associated with another installation method. Consideration shall be given to accidental flooding of compartments and the implication of this for other platform parts.

The flooding system, the buoyancy compartments and any necessary lifting connections on the structure shall be designed to ensure that the structure can be safely uprighted and landed on the sea floor.

22.2.6.2 Flooding controls

The location and accessibility of all controls for selective flooding and righting, as well as the protection of the controls from environmental and operational hazards, shall be carefully considered.

22.2.6.3 Model tests and analysis

Model tests and detailed calculations shall be considered for any platform part to be towed to site, in order to determine towing and stability characteristics during towing and upending procedures.

22.3 Transfer of the structure from the transport barge into the water

22.3.1 Lifting operations

For all lifting operations, the strength of the structure and the suitability of lifting equipment, including acceptable margins on capacity and reach, shall be determined. The actions shall be derived in accordance with Clause 8 and members and joints checked to ensure they are adequate for the lift conditions.

Lifting operations shall be performed in accordance with the agreed plans. The operations should not be performed under more severe environmental conditions than those considered in the design phase.

Prior to lifting, the lifted weight shall be assessed to ensure that it is within the limits defined by the design and within the capacity of all lifting equipment. If no weighing is carried out, an adequate margin should be applied to cover mill tolerances and growths in piping, equipment weights, etc.

The derivation of design actions for lifting points is addressed in Clause 8. Padeye plates should be oriented in such a direction that the possibility for out-of-plane actions on the padeye plate and shackle are minimized.

The rigging shall be designed to carry actions derived in accordance with 8.7, with the appropriate resistance requirements according to this International Standard to allow the structure to be lifted off the barge and lowered into the water. The slings should be attached above the centre of gravity of the structure being lifted to provide good control of the operation.

22.3.2 Launching

22.3.2.1 General

For those structures that are to be launched, a launching system shall be provided considering the items listed in subclauses 22.3.2.2 to 22.3.2.5.

22.3.2.2 Launch barge

The launch barge shall be equipped with launchways, rocker arms, a controlled ballast and dewatering system, and an appropriate power unit (hydraulic ram, winch, etc.), to assist the structure to slide down the ways.

22.3.2.3 Actions on the structure

The structure to be launched shall be designed and fabricated to withstand the actions of the launch. This can be done by either strengthening those members that can become overstressed by the launching operation or by designing a special frame into the structure, commonly referred to as a launch frame. A combination of the above two methods may also be used.

22.3.2.4 Flotation

A structure that is to be launched shall be watertight and buoyant. If its upending is to be assisted by a crane barge, the launched structure should float in a position such that lifting slings from the crane can be safely attached. The attachment points of any pre-installed slings should be exposed and accessible.

22.3.2.5 Crane barge requirements

The crane barge shall be of sufficient size

a) to change the position of the launched structure from its floating position to its upright position, or

b) to hold the launched structure at the site until it can be righted by a controlled flooding system.

22.4 Placement on the sea floor and assembly of the structure

22.4.1 General

Placement on the sea floor and assembly of the structure shall be in accordance with the installation plan.

22.4.2 Safety of navigation

Safety and navigational aids shall be positioned and maintained as required throughout the installation phase, particularly when the structure is to be left for some time before the installation of the topsides.

22.4.3 Anchoring

During marine operations, the crane and cargo barges may be held in position using either dynamic positioning systems or anchors.

Where anchors are used, the following applies, in accordance with ISO 19901-7[7].

a) The length of anchor lines shall be adequate for the water depth at the site.

b) Anchor sizes and shapes shall be selected such that the anchors will bite and hold in the seabed at the site. The holding capacity should be sufficient to resist the strongest tides, currents and winds that can reasonably be expected to occur at the site.

22.4.4 Positioning of the structure

22.4.4.1 General

The term *positioning* includes the placement of the structure on the sea floor at the installation location, in a position and with an orientation within the allowable tolerances, and levelling the structure ready for the installation of the foundation piles, in accordance with the installation plan. This often requires upending the platform parts that have been towed to the site or launched from a barge at the site.

If the structure is to be installed over one or more existing wells, the wellheads shall be protected from damage through accidental contact with the structure. Advance planning and preparation shall be in such detail as to minimize hazards to both the wells and the structure. If the structure is not to be installed over an existing well, and not to be located adjacent to an existing structure, parameters for the accuracy of the positioning should be stated in the installation plan.

22.4.4.2 Upending

The upending process is generally accomplished by a combination of crane barge assistance and controlled or selective flooding systems. The upending phase requires advanced planning to predetermine the simultaneous lifting and controlled flooding steps necessary to set the structure on the sea floor. Closure devices, lifting connections, etc. shall be provided where necessary. The flooding system shall be designed to withstand the water pressures that will be encountered during the positioning process.

22.4.4.3 Levelling of the structure

The structure shall be levelled within the tolerances specified in the installation plan. Levelling the structure before any of the piles are installed is preferred, but often not practical. It can therefore be necessary to level the structure by jacking or lifting after a minimum number of piles have been driven. Levelling the structure after all the piles have been installed should be avoided if possible. Procedures shall minimize bending stresses in the piles. After the structure has been levelled, care should be exercised to maintain its levellness during further pile installation operations.

22.4.4.4 Structure on-bottom weight

The soil stresses under foundation components at the base of the structure (mudmats, footings or other bearing components) prior to the installation of the permanent pile foundation can be critical. The distribution of the structure's weight on the sea floor shall be considered for each combination of pile sections that are planned to be supported from the structure. Bearing capacity analyses shall be performed in accordance with ISO 19901-4.

For soils that increase in strength with depth, particularly soft clays and loose sands, the bearing capacity analysis should account for shape effects and for the presence of any holes in the mudmats. This is because any reduction in mudmat area can result in a potential shallow failure surface and hence a reduced bearing capacity. Also, if there are loose sands at the sea floor, vibrations produced by pile driving operations can contribute to additional settlement.

The increase in soil stresses resulting from waves of the maximum height anticipated during the installation period shall be considered. The bearing capacity analysis shall take account of the combined effect of vertical and horizontal action effects (forces and moments) on foundation components at the base of the structure. The more heavily loaded components can experience a reduction of soil stiffness which can allow forces to be transferred to other components. In the analysis, consideration may be given to suction developing under components that are subjected to uplift, provided that the components have been designed with an adequate skirt length and that measures have been taken to prevent ingress of sea water into the skirt compartments, e.g. by the provision of valves.

The structure and foundation components at its base should be checked for the applicable actions specified in 8.7.6. The environmental action (E_e) in Equation (9.10-1) should be replaced by the action determined from waves of the maximum height anticipated during the installation period.

In the event of rough seas, or if the installation equipment leaves the site for other reasons before the structure has been adequately secured with piles, it can be necessary to adjust the effective weight on bottom to minimize the possibility of movement of the structure due to skidding, overturning or soil failure.

22.5 Pile installation

22.5.1 General

Proper installation of piling, including conductor piles, is vital to the life and permanence of the structure and requires each pile to be driven to or near design penetration, without damage. All field-made structural connections shall be compatible with the design requirements. Pile sections should be marked in a manner to facilitate installing the pile sections in the proper sequence. The closure device on the lower end of the structure's legs and pile sleeves, if required, shall be designed to avoid interference with the installation of the piles.

The pile wall thickness shall be adequate to resist axial and lateral actions as well as the stresses during pile driving (see 17.10). The stresses resulting from pile driving can be predicted approximately from computer analyses based on the principles of one-dimensional elastic stress-wave theories, run as part of driveability studies (see 22.5.5).

22.5.2 Stabbing guides

Add-on pile sections shall be provided with guides to facilitate stabbing and alignment. The guide should provide a tight and uniform fit for proper alignment. The guides shall be capable of safely supporting the full weight of the add-on pile section prior to welding.

22.5.3 Lifting methods

If lifting padeyes are used for the handling of the pile sections, the padeyes shall be designed (with due regard for the influence of impact) for the stresses developed during the initial pick-up of the section as well as those occurring during the stabbing of the section. If lifting padeyes or weld-on lugs are used to support the initial pile sections from the top of the structure, the entire hanging weight should be considered to be supported by a single padeye or lug. The lifting padeyes or support lugs should be removed by torch cutting 6 mm (0,25 in) from the pile surface and ground smooth. Care should be exercised to ensure that any remaining protrusion does not prevent driving of the pile or cause damage to elements such as packers. If burned holes are used in lieu of lifting padeyes, they should comply with the applicable requirements of this subclause, and consideration should be given to possible detrimental effects during hard driving.

As an alternative to providing lifting padeyes on the piles, pile handling tools may be used, provided they are of the proper size and capacity for the piles being driven and the operating conditions anticipated. The tools should be inspected prior to each use, to ensure that they are in proper working condition. They should be used in strict accordance with the manufacturer's instructions and/or recommendations. For installations that require the use of pile followers, the followers should be inspected prior to their first use and periodically during the installation, depending on the severity of pile driving.

22.5.4 Field welds

The add-on pile sections should be carefully aligned and the bevel inspected in order to assure that a full penetration weld can be obtained before welding is initiated. If necessary, the bevel should be opened up by grinding or gouging. Welding should be performed in accordance with Clause 20. Non-destructive testing of the field welds, using one or more of the methods referenced in Clause 20, should be performed.

22.5.5 Driveability studies

Computer analyses (commonly known as wave equation analyses) can be used to simulate the hammer-pile-soil system and pile driving behaviour, with the objective of defining the range of blow counts necessary to reach the target design pile penetration and assessing the stresses in the pile resulting from pile driving. The predicted range of blow counts to reach a given penetration is governed by the assumed profile of soil resistance to driving (SRD), but will also be affected by the assumed hammer efficiency, or energy transferred to the pile and, to a lesser extent, by the so-called quake and damping parameters in the wave equation model. Therefore, selection of these input parameters should be based on previous pile driving experience and engineering judgment. The energy transferred to the pile is highly dependent on the type of hammer (i.e. diesel, steam, or hydraulic hammer) for a given rated energy and should be based on pile driving experience with reliable measurements from pile instrumentation.

The definition of SRD is the main factor that governs the results of the driveability studies, i.e. the hammer type required to reach the target pile penetration. Several methods for calculating the SRD in different types of soil have been proposed in the literature, see A.22.5.5.

General procedures cannot be applied at all sites, as piling behaviour is highly site-dependent. Therefore, back-analysis of previous pile driving experience at the site, or at sites with similar soil conditions, is recommended in order to calibrate SRD calculation procedures and improve driveability predictions for other structures at the site. For cohesive soils, the SRD calculation should take into account the increase in resistance due to pore pressure dissipation (set-up) during delays, in particular when delays are necessary for welding add-on elements.

To confirm that the hammer performs in accordance with the specifications and with the assumptions made in the driveability predictions, the pile/hammer should be instrumented and monitored during driving. Pile instrumentation is preferable, as hammer monitoring does not provide sufficient information about the driving energy actually transferred into the pile.

Pile driving instrumentation data, based on measurements from strain and acceleration transducers fixed near the top of the pile, may be used for verifying pile hammer efficiency, soil stratification, assessing the actual SRD during driving, as well as giving additional information for estimating the ultimate static pile capacity — particularly if re-strike test data are available. The actual SRD during driving, as back-calculated from pile instrumentation data, should be compared with the predicted range in soil resistance. Such analyses can be used to improve the reliability of subsequent driveability predictions at the site.

22.5.6 Obtaining required pile penetration

The adequacy of the structure's foundation depends upon each pile being driven to or near its design penetration. Where applicable, the driving of each pile should be carried to completion with as little interruption as possible to minimize the increased driving resistance which often develops during delays. It is often necessary to work one pile at a time during the driving of the final one or two sections, so as to minimize set-up time. Workable back-up hammers with leads should be available, especially when critical pile set-up is anticipated.

The fact that a pile has met premature refusal does not assure that it is capable of supporting the design actions. Final blow count alone cannot be considered as assurance of piling adequacy. Continued driving beyond the defined refusal (see 22.5.7) can be justified if it offers a reasonable chance of significantly improving the capacity of the foundation without risk of damaging the pile, hammer or structure.

In some instances, when continued driving is not successful, the penetration and associated capacity of a pile can be improved by the methods described in 22.5.8. Such methods should be approved by the design engineer.

22.5.7 Driven pile refusal

Pile refusal requires defining, primarily in order to

a) establish the point at which pile driving with a particular hammer should be stopped and other methods instituted (see 22.5.8), and

b) prevent damage to the pile and hammer.

The definition of refusal should be consistent with the individual soil characteristics anticipated at the specific location. Refusal should be defined for all hammer sizes to be used and is contingent upon the hammer being operated at the energy and rate recommended by the manufacturer.

In establishing the pile driving refusal criteria, the recommendations of the pile hammer manufacturer should be considered. The exact definition of pile refusal for a particular installation should be defined in the installation specification. Examples of refusal criteria (for use only in the event that no other requirements are included in the installation specification) are given in A.22.5.7.

22.5.8 Pile refusal remedial measures

If a pile refuses before it reaches design penetration, one or more of the following measures can be taken.

a) Review of hammer performance

A review of all aspects of hammer performance, possibly with the aid of hammer and/or pile head instrumentation, can identify problems that can be solved by improved hammer operation and maintenance, or by the use of a more powerful hammer.

mm) Re-evaluation of design penetration

Reconsideration of actions, displacements and required capacities of individual piles, of other foundation elements and of the foundation as a whole, can identify available reserve capacity. An interpretation of driving records in conjunction with instrumentation, see a) above, can allow design soil parameters or stratification to be revised and the calculated pile capacity to be increased.

b) Modifications to piling procedures

Modifying procedures, usually the last course of action, can permit the piles to be driven to the required penetration after all. The following modifications can be considered.

1) Plug removal

The soil plug inside the pile can be removed by jetting and air lifting, or by drilling, to reduce pile driving resistance. Several soil plug removals and redrives can be required to reach target penetration.

If plug removal results in inadequate pile capacity, the removed soil plug shall be replaced by a grout or concrete plug. The minimum axial capacity of the plug shall be equal to the pile end bearing capacity in a plugged condition. Attention shall be paid to the characteristics of shear transfer between plug and pile. In some circumstances plug removal is not effective, particularly in cohesive soils.

2) Soil removal below the pile tip

Soil below the pile tip can be removed, either by drilling an undersized hole or by jetting and possibly air lifting. The drilling or jetting equipment is lowered through the pile, which acts as the casing pipe for the operation.

Considering the resulting uncertainties with respect to the pile axial capacity, the soil below the pile tip should not be removed to reduce the soil resistance during driving in uncemented soils.

Under special circumstances, e.g. in the case of an intermediate layer of strong cemented material, undersized drilling can be applied to partially remove the hard layer before pile driving can be resumed. The depth of drilling should be restricted to the thickness of the hard cemented layer.

Undersized drilling should be restricted to relatively thin and not too hard layers. In thick and very hard rock layers preferably under-reaming of the hole to at least the full pile size should be applied to avoid potential risk of pile tip buckling.

Where soil removal below the pile tip has been performed by drilling (undersized or otherwise), the contribution of the relevant zone of soil to the pile capacity should be ignored, unless this zone has been grouted.

Jetting below the pile tip should, in general, be avoided because of the unpredictability of the results.

3) **Two-stage driven piles**

A first stage or outer pile can be driven to a predetermined depth, after which the soil plug is removed, and a second stage or inner pile is driven inside the first stage pile. The annulus between the two piles is grouted to permit shear transfer between the first and second stage piles and to develop composite action.

4) **Drilled and grouted piles and belled piles**

See 22.5.10 and 22.5.11, also 17.2.3 and 17.2.4.

In order to minimize delays in installation, a pile acceptance procedure should be established. The procedure should outline the measures to be taken on location for adjusting planned pile driving scenarios, in case of, for example, premature pile driving refusal or a significantly lower blow count than anticipated at design target pile penetration.

22.5.9 Selection of pile hammer

If piles are to be installed by driving, the influence of the hammers to be used shall be evaluated as part of the design process in accordance with 17.10. The type(s) of pile hammer considered for use during driving shall be noted by the designer on the installation drawings or specifications.

Any change in the hammers to be used for pile driving shall be assessed, in order to ensure that the consequences of the change are acceptable (including pile drivability, pile capacity, pile and structure strength and fatigue).

22.5.10 Drilled and grouted piles

The hole for drilled and grouted piles may be drilled with or without drilling mud to facilitate maintaining an open hole. Drilling mud can be detrimental to the surface of some soils. If used, consideration should be given to flushing the mud with circulating water upon completion of drilling, provided the hole will remain open.

Reverse circulation should normally be used to maintain sufficient flow for removal of cuttings. Drilling operations should be done carefully, in order to maintain proper hole alignment and minimize the possibility of hole collapse. An insert pile with an upset drill bit on its tip may be used as the drill string, so that it can be left in place after completion of the hole.

Centralizers should be attached to the pile to provide a uniform annulus between the insert pile and the hole. A grouting shoe may be installed near the bottom of the pile to permit grouting of the annulus without grouting inside the pile. If a grouting shoe is used it can be necessary to tie the pile down to prevent flotation in the grout. The time before grouting the hole should be minimized in soils which may be affected by exposure to sea water. The quality of the grout should be tested at intervals during the grouting of each pile. Means should be provided for determining that the annulus is filled. Holes for closely positioned piles should not be open at the same time unless there is assurance that this will not be detrimental to pile capacity and that grout will not migrate during placement to an adjacent hole.

22.5.11 Belled piles

Drilling of the bell is carried out through the pile by under-reaming with an expander tool. A pilot hole may be drilled below the bell to act as a sump for unrecoverable cuttings. The bell and the pile are filled with concrete to a height sufficient to develop the necessary transfer of forces between the bell and the pile. Bells are connected to the pile to transfer both full uplift and compressive forces using steel reinforcing such as structural members with adequate shear lugs, deformed reinforcement bars or prestressed tendons. In general, drilling of bells for belled piles should employ only reverse circulation methods. Drilling mud should be used where necessary to prevent caving and sloughing. The expander or under-reaming tool used should have a positive indicating device to verify that the tool has opened to the full width required. The shape of the bottom surface of the bell should be concave upward (sides higher than the centre) to facilitate later filling of the bell with tremie concrete.

To aid in concrete placement, longitudinal bars and spiral steel reinforcements should be well spaced. Reinforcing steel may be bundled or grouped to provide larger openings for the flow of concrete. Special care should be taken to prevent undue congestion at the throat between the pile and the bell, where such congestion can trap laitance. Reinforcing steel cages or structural members should extend far enough into the pile for an adequate transfer of forces to be developed.

Concrete should be placed as for tremie concrete, with the concrete being ejected from the lower end of a pipe at the bottom of the bell, always discharging into fresh concrete. Concrete with aggregates of 10 mm (3/8 in) and less in size may be placed by direct pumping. Because of the long drop down along the pile and the possibility of a vacuum forming with subsequent clogging, an air vent should be provided in the pipe near the top of the pile. To start placement of concrete, the pipe should have a steel plate closure with soft rubber gaskets in order to exclude water from the pipe. Care should be taken to prevent unbalanced fluid heads and a sudden discharge of concrete. The pile should be filled to a height above the design concrete level equal to 5 % of the total volume of concrete, placed so as to displace all laitance above the design level. Suitable means should be provided to indicate the level of the concrete in the pile.

Concrete placement in the bell and adjoining section of the pile should be as continuous as possible.

22.5.12 Grouting pile-to-sleeve connections and grouted repairs

The grouting operation should be performed using an approved and proven procedure by experienced grouting operatives.

The equipment should have sufficient capacity to achieve the grout filling in a single continuous operation. Grouting should not commence unless there are sufficient materials available, including a contingency, to complete the task. Grout slurry stored in a holding tank should be continuously stirred and should not be held for more than 30 min prior to pumping. In case of rapid hardening mixes, the storage duration in a holding tank should be reduced to less than 30 min.

Prior to grouting and after activating any sealing devices, dyed water should be flushed through the complete grouting system to both remove any deleterious matter and to prove its functionality. A pressure test can be appropriate for closed systems. The annulus should then be carefully filled by maintaining a continuous grout flow through the lowest practical point.

Grout returns to allow surface sampling are preferable. If these are provided, grout samples for strength compliance testing may be taken from the returns in addition to the slurry specific gravity measurements. If surface returns are not provided, visual inspection to confirm that grout has completely filled the annulus should be performed immediately after cessation of grout pumping and again after initial grout set, typically 12 h.

22.5.13 Pile installation records

Throughout the driving of main or skirt piles, comprehensive driving and associated data should be recorded and reviewed for conformance with the installation plan. If significant deviations are observed, it can be necessary to take appropriate measures. The recorded data can include, as appropriate,

a) structure and pile identification, water depth and reference elevation of readings,

b) relevant information on pile stabbing,

c) penetration of the pile under its own weight or under the weight of a new add-on,

d) penetration of the pile under the weight of the hammer,

e) where applicable, data on followers used,

f) blow counts throughout driving, with hammer identification and energy observations,

g) at relevant penetrations, the cumulative number of blows,

h) the hammer blow rate (blows/minute) after every few metres of penetration,

i) if available, pile instrumentation data,

j) unusual behaviour of the hammer or the pile during driving,

k) date and time of starts and stops in driving, including set-up time,

l) elapsed time for driving each section,

m) elevations of soil plug and internal water surface after driving,

n) actual length of each pile section and cut-offs, and

o) pertinent data of a similar nature covering driving, drilling, grouting or concreting of grouted or belled piles.

22.5.14 Use of hydraulic hammers

Hydraulic hammers tend to be more efficient than steam hammers and the energy transferred to the pile for a given rated energy tends to be greater. They can be used both above and below water for driving battered or vertical piles, through legs or through sleeves and guides, as well as vertical piles through sleeves alone. In calculating pile stresses, full account should be taken of wave, current and wind actions, both during driving and during hammer stabbing (which can be either above or below water). While for steam hammers the weight of the cage is generally held by a crane, for hydraulic hammers the whole weight of the hammer is borne by the pile.

The energy output is generally varied by the installation contractor to maintain a fairly low blow count. Therefore, blow counts do not give a direct guide to soil stratification and resistance. Since the ram is encased, hammer performance cannot be judged visually. It is therefore important that measurements be made to give a record of the hammer's performance, including ram impact velocity, stroke, pressure of accelerating medium, and blow rate. Reliable instrumentation of some piles should also be considered, to verify the energy transferred to the pile to aid interpretation of soil stratification and to limit pile stresses.

Monitoring of underwater driving requires that easily identified, unambiguous datum points be used, together with robust television cameras or remotely operated vehicles capable of maintaining station. Alternatively, for shallow water sites, it is possible to extend the hammer casing so that blow counts can be monitored above water.

Because no cushion block is used, there is no change in characteristics between ram and anvil as driving progresses and no requirement for cushion changes. However, because of the steel-to-steel contact, particular attention should be paid to the design of the pile head.

In selecting hydraulic hammers for deeper water applications, account should be taken of possible decreases in efficiency due to increased friction between the ram and its surrounding air. Sufficient air should be supplied to the hammer so that water ingress is prevented. Water in the pile should be able to escape freely. It should be noted that hydraulic hammer changes take much longer than steam hammer changes.

22.6 Installation of conductors

The planning and execution of conductor installation and shallow well drilling should recognize the potential for disturbance to foundation soils and the consequent risk of a reduction in stability of the fixed structure or of adjacent conductors.

During drilling operations, soil disturbances can result from hydraulic fracture, from wash-out or from encountering shallow gas pockets. Hydraulic fracture occurs where drilling fluid pressure is too high and fluid is lost into the formation, possibly softening the surrounding soil. Wash-out (uncontrolled enlargement of the drilled hole) generally occurs in granular soils and can, in part, be induced by high drilling fluid circulation rates. Wash-out leads to stress relief in the surrounding soils. These incidents can be accompanied by loss of circulation of drilling fluids, by return of these fluids to the seabed other than through the conductor, or by the creation of seabed craters.

If piles are installed within the zone of influence of soil disturbance, reduction in axial or lateral capacity and foundation stiffness can occur. Similarly, the stability of shallow foundations can be reduced and settlements increased.

It should be noted that these detrimental effects can occur whether the drilling takes place either after installation of the structure or before, e.g. for a pre-installed template or for an exploration well.

The proximity of conductor slots to as-installed or future pile locations is critical and risks are clearly greater for narrow structures with vertical piles.

The following recommendations should be considered for conductor installation and shallow well drilling.

a) The conductor setting depth should be selected taking due account of hydraulic fracture pressure profiles. The depth should preferably be chosen at a cohesive stratum which is a sufficient distance from the proposed pile tip penetration to minimize the risk of disturbance of foundation soils.

b) Particular care should be exercised in the installation of conductors which are installed by drilling or drill-drive techniques instead of by driving alone.

c) In conductor or shallow well drilling operations, fluid pressures should be kept within the calculated hydraulic fracture pressure profile. Flow rates should be controlled to minimize wash-out, particularly in granular soils.

d) Records of conductor installation and shallow well drilling should be available to the structure's design engineer. The implications for foundation soils of any incidents of excessive loss of circulation, of return of drilling fluids to the seabed other than through the conductor, or of creation of seabed craters should be assessed.

22.7 Topsides installation

22.7.1 General

The topsides installation will normally consist of lifting items such as deck sections, module support frames, modules and packages from the transport barges onto the structure, connecting them to the structure and to each other as specified by the design.

22.7.2 Alignment and tolerances

After the piling has been driven and cut off to grade, the topsides should be set, with proper care being exercised to ensure proper alignment and elevation. The topsides parts shall be aligned within the tolerance specified in the design documents. Unless otherwise specified, the deck elevation shall not vary more than ± 75 mm (3 in) from the design elevation shown in the drawing. The finished elevation of the deck shall be within 13 mm (0,5 in) of level.

22.7.3 Securing topsides

Once the topsides parts have been set, they should be secured to provide the support and fixity as required by the design.

22.7.4 Appurtenances

Once the topsides are installed, all stairways, handrails, and other similar appurtenances should be installed as specified.

22.8 Grounding of installation welding equipment

22.8.1 General

Normal welding procedures use reverse polarity, wherein the welding rod is positive and the ground is negative. An adequate and properly placed ground wire is necessary to prevent stray currents, which, if uncontrolled, can cause severe corrosion damage.

22.8.2 Welding equipment

The welding machine should be located on, and grounded to, the structure whenever possible. When this is impossible or impractical, and the welding machine is located on the barge (or vessel), both leads from the output of the welding machine should be run to the structure and the ground lead secured to the structure as close as practical to the area of welding. Under no conditions shall the hull of the barge (or vessel) be used as a current path. The case or frame of the welding machine shall be grounded to the hull to eliminate shock hazards to personnel.

The welding cables should be completely insulated to prevent stray currents. Damaged cables shall not be allowed to hang in the water.

Grounding cable lugs should be tightly secured to grounding plates. The contact should be thoroughly cleaned to bare metal. The resistance of the connection should be a maximum of 125 μΩ per connection or the voltage drop across the connection should be a maximum of 62,5 mV for a current of 500 A.

The minimum cross-sectional area of the return ground cable should be 645 mm^2/1 000 A per 30 m (100 ft) of cable. One or more cables connected in parallel may be used to meet minimum cross-section requirements.

More than one ground cable of sufficient size should be used to guard against a single return or ground becoming loose.

Connecting several welding machines to a common ground cable which is connected to the structure being welded will control stray currents if adequately sized and properly insulated from the barge (or vessel) containing welding machines.

22.8.3 Monitoring remote ground efficiency

If welding is conducted using generators remote from a structure, grounding efficiency can be monitored by simultaneously measuring the potential of the structure and the barge (or vessel) housing the welding generators. A change in potential reading from either indicates insufficient grounding.

23 In-service inspection and structural integrity management

23.1 General

23.1.1 Applicability

This clause defines in-service inspection requirements for both the underwater and above water parts of fixed steel offshore structures located anywhere in the world, built to any design and fabrication standard, and of any age. In addition the structural integrity management requirements relating the inspection results to ongoing requirements for inspection, repairs and maintenance are given.

This clause includes inspection requirements for members, joints and connections associated with

— primary structural framing,

— leg/pile connections,

— external inspection of pipeline risers and supports,

- external inspection of J-tubes and supports,
- conductor guide framing,
- service caissons and supports,
- riser guards,
- boat landings and fenders,
- other secondary framing and appurtenances.

Also addressed is in-service inspection of the underwater cathodic protection systems and any special corrosion coatings of components located in the splash zone.

Structural integrity management (SIM) is a structured method recommended for assuring the condition of a structure and is a cyclic activity covering

- data collection,
- data evaluation,
- development of an inspection strategy,
- development and execution of an inspection programme, and
- any consequent remedial works.

A default inspection programme that may be used in the absence of a more specific SIM system is provided. Intervals for underwater inspections may be extended beyond the requirements of the default inspection programme, provided the owner can show, through a specific SIM system, that a structure or group of similar structures are fit-for-purpose during the interval to the next inspection.

23.1.2 Inspection motives

The purpose of in-service structural inspection is to determine, with a reasonable level of confidence, the existence and extent of deterioration, defects, or damage. Data collected during an inspection are needed to verify the integrity of the structure. The following are motives for in-service inspection of fixed steel offshore structures:

- fabrication defects or installation damage;
- degradation or deterioration of the structure;
- design uncertainties or errors;
- environmental or weight overload;
- accidental events;
- changes in permanent actions;
- monitoring of known defects or repair effectiveness;
- change of ownership;
- statutory requirements;
- reuse.

23.1.3 Inspection in structural integrity management

In-service inspection is an integral part of structural integrity management, which is an ongoing process for ensuring the fitness-for-purpose of an offshore structure or group of structures. The four phases of this process are shown in Figure 23.1-1. The phases shown in Figure 23.1-1 are described in 23.2 to 23.5. Inspection requirements are given in 23.6. Structural integrity management can allow the owner to relax the default inspection requirements described in 23.7.

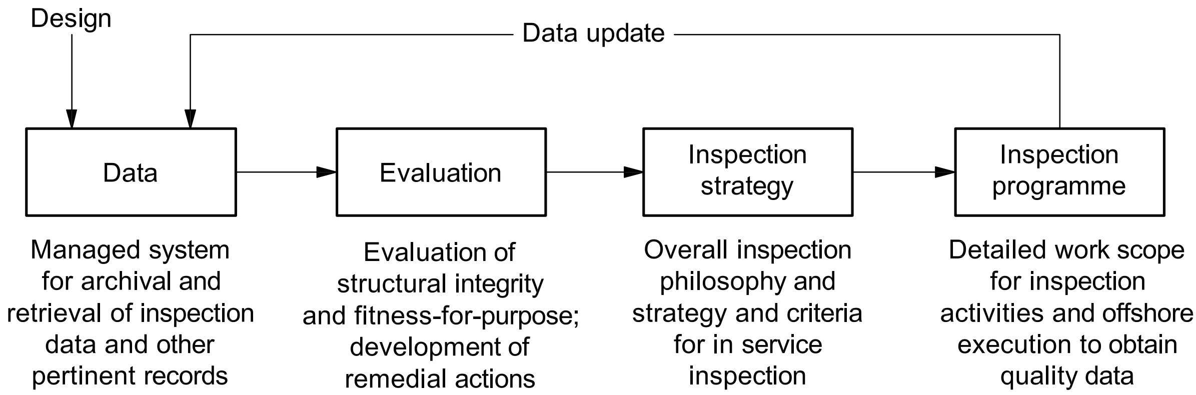


Figure 23.1-1 — Phases of a structural integrity management cycle

23.2 Data collection and update

Essential aspects of structural integrity management are the validity, extent and accuracy of the structure's data and inspection history. Accordingly, records of all original design analyses, fabrication, transportation, installation (including piling) and in-service inspections, engineering evaluations, repairs, and incidents shall be retained by the owner for the life of the structure and transferred to new owners as necessary.

23.3 Evaluation

23.3.1 General

When additional structure data or new inspection data become available or when the inspection strategy is being updated to develop an inspection programme, all available data on the structure or group of structures shall be evaluated using engineering judgment, operational experience, analysis and predictive techniques, as appropriate. An assessment (see Clause 24) is required if the evaluation of relevant SIM data determines that an assessment initiator (24.4) has been triggered.

The evaluation can conclude either

a) that the structure is fit-for-purpose between inspections and only requires on-going inspection (with a specified scope), or

b) that remedial measures (immediate or longer term) are required.

23.3.2 Risk assessment

The evaluation, consequent inspection strategy and any remedial measures shall meet the objectives of the SIM system in maintaining fitness-for-purpose, and shall be based on consideration of the consequences of structural component failure and perceived likelihood of such failure.

Risk matrices in combination with the platform exposure levels determined from 6.6 are useful tools, but should be supplemented with more detailed risk assessments if the overall exposure level is too coarse or too general to address specific potential concerns, aspects of performance or individual components.

Probability-based inspection methods can provide information on inspection priorities but shall be used with care, due to uncertainties in the input data and the degree of uncertainty of fatigue predictions.

23.3.3 Structural considerations in evaluation

Evaluation requires consideration of numerous factors affecting the structural performance and corrosion protection for the various structural components. The following structural performance factors shall be considered, as appropriate:

a) structure age, condition, original design situations and criteria, and comparison with current design situations and criteria;

b) analysis results and assumptions for original design or subsequent assessments;

c) structure reserve strength and structural redundancy;

d) fatigue sensitivity;

e) degree of conservatism or uncertainty in specified environmental conditions;

f) extent of inspection during fabrication and after transportation and installation;

g) fabrication quality and occurrences of any rework or rewelding;

h) damage (including fatigue damage) during transportation or installation;

i) operational experience, including previous in-service inspection results and lessons from the performance of other structures;

j) modifications, additions and repairs or strengthening;

k) occurrence of accidental and severe environmental events;

l) criticality of structure to other operations;

m) structure location (geographical area, water depth);

n) debris;

o) structural monitoring data, if available; and

p) potential reuse or removal intents.

23.3.4 Corrosion control considerations in evaluation

Aspects of corrosion control that shall be considered in the evaluation include

— the design situations, assumptions and criteria used,

— the details of the system (e.g. active or passive and the materials from which sacrificial anodes are made), and

— the past performance of the system.

Further requirements for the inspection and monitoring of corrosion control, particularly for active systems, are given in 18.6.

23.3.5 Remedial measures

If the risk assessment results are unacceptable, remedial measures which reduce the risk of structural failure, whether affecting likelihood or consequence, shall be implemented. The choice of individual or combinations of remedial measures and their extent will depend on the source of risks to structural integrity and their magnitude.

23.4 Inspection strategy

23.4.1 Basis

The in-service structural inspection strategy should be developed from the evaluation (see 23.3) and should consider statutory requirements, owner's corporate policy, and industry or national standards and practices as appropriate. The inspection strategy should be developed for each structure during or soon after design, revised as necessary after fabrication and installation, and periodically updated throughout its service life (usually through amendments following the receipt of additional inspection reports, results of structure reanalyses or assessments, and other data or information pertinent to the structural integrity management system).

23.4.2 Requirements

An inspection strategy shall

a) as a minimum, determine within a reasonable level of confidence the existence and extent of deterioration, defects and damage,

b) consider the interdependence between the four phases of the SIM system shown in Figure 23.1-1,

c) address the motives for inspection (see 23.1.2) and structural integrity management,

d) consider the factors discussed in 23.2.4 and 23.3.3,

e) include the inspection types listed in 23.4.3,

f) specify the inspection tools and techniques (see 23.4.5) to be used,

g) specify any remedial measures (including tools, methods and any constraints) to be taken immediately (e.g. remedial grinding),

h) be developed and maintained by competent personnel in accordance with 23.8,

i) be developed using experience, data, and/or analysis or predictive technology, and

j) be documented.

NOTE National and regional regulations can require a SIM system to be documented in a form suitable for verification or for review by the regulator.

23.4.3 Inspection types

The inspection strategy should contain different inspection types, as listed below and as defined in 23.6.

a) Scheduled inspections

1) Baseline

Baseline inspections are conducted to determine the initial condition of the structure for use as a benchmark for items not included in fabrication and installation inspections, and to detect any transportation or installation damage and any early appearance of defects or deterioration. The most important factor affecting the scope of work for this inspection is the extent of inspection performed during fabrication and installation (see 21.4).

2) Periodic

Periodic inspections are conducted to assure structural integrity by detecting any deterioration and degradation over time and discovering any defects. The key aspects of a periodic inspection strategy are the inspection interval and the scope of work, both of which can vary depending on the results of periodic evaluations during the **life** of the structure. Non-destructive testing (NDT) can be required for fatigue-sensitive structures.

There are four levels of periodic inspection. Each level is detailed according to the type, scope and detail of the inspection, as described in 23.6.

1) **Special**

Special inspections are conducted to monitor repairs, remediation programmes, known damage and defects, or known areas of vulnerability (underdesign, scour, etc.). Special inspections can also be needed for structure reuse (see Clause 25). The key features of special inspections are definition of the objectives, selection of appropriate tools and techniques, scopes of work, and inspection intervals.

b) Unscheduled inspections (reacting to events)

Unscheduled inspections are conducted to evaluate a structure's condition following

— an environmental event (e.g. storm, earthquake, mudslide), or

— an incident (e.g. vessel impact, dropped object, explosion).

All unscheduled inspections shall be developed based on an evaluation of the available data (see 23.3), including any event or incident reports.

While the timing of an unscheduled inspection may, subject to evaluation, be advanced or delayed to coincide with a scheduled inspection, a separate inspection programme can be required to provide data to confirm adequate structural integrity or to allow repair/strengthening work to be designed and planned to fit seasonal weather windows.

The extent of inspection during fabrication and installation has a direct bearing on the in-service inspection strategy, since it influences what is known about the initial condition of the structure and the quality of its construction.

23.4.4 Factors to consider in determining strategy

Factors to consider in developing an inspection strategy shall include those listed in 23.3.3 and should also include the following:

a) scheduling flexibility, including

— intervals between periodic inspections, and

— promptness of post-event and post-incident inspections;

nn) cost, capability and availability of inspection equipment and services, including

— inspection tools and specialized equipment,

— personnel,

— deployment and support vessels and equipment,

— seasonal weather windows;

oo) regional differences, such as those resulting from environmental differences, including

— the severity and frequency of storms,

— conditions relevant for fatigue,

— seismicity levels,

— wind speeds, and/or

— the presence of sea ice and icebergs;

pp) reliability and applicability of inspection technique(s), e.g. probability of detection and accuracy of sizing, which should be considered with due regard to the type of data required and the sensitivity of the structure to a particular form of damage.

23.4.5 Inspection methods and applications

Selection of appropriate inspection method(s) for a given task shall be determined by qualified personnel (see 23.8) based on the goals/objectives of the inspection and the sensitivity, effectiveness, reliability, cost, and availability of the various tools and techniques. The types of survey and specific techniques are discussed in A.23.4.5.

23.5 Inspection programme

The inspection programme is the detailed scope of work for the offshore execution of inspection activities. The inspection programme is developed from the inspection strategy.

While the general scope of work for the inspection programme follows from the inspection strategy, the programme requires schedules, budgets, personnel profiles and other procedures before it can be implemented.

If, during the course of an inspection programme, any deterioration, defects, damage or other anomalies are discovered that could potentially affect the structural integrity or fitness-for-purpose of the structure, conductors, risers and J-tubes, or appurtenances, these shall be evaluated to determine if and when additional inspection and/or remedial measures should be undertaken. Additional inspection can require the use of more specialized or sensitive inspection techniques than employed during discovery.

Additional inspection shall be performed as soon as conditions permit, preferably during the same programme of work.

All deterioration, defects, damage or other anomalies, and any follow-up action shall be documented, with all records and reports retained.

23.6 Inspection requirements

23.6.1 Baseline inspection

A baseline inspection shall be conducted to establish the as-installed condition as soon as practical after the installation and commissioning, and, wherever possible, within the first year of operation. This baseline inspection is required to inform the evaluation (see 23.3) on which the inspection strategy (see 23.4) is based. The minimum scope shall consist of

a) a visual inspection without marine growth cleaning that provides full coverage of the structure (members and joints), conductors, risers, and various appurtenances, and which includes benchmarking the sea floor conditions at the legs/piles and checking for debris and damage,

b) a set of cathodic potential readings that provides full coverage of the underwater structure (members and joints), conductors, risers, and various appurtenances,

c) visual confirmation of the existence of all sacrificial anodes, electrodes, and any other corrosion protection material/equipment,

d) measurement of the actual mean water surface elevation relative to the as-installed structure, with appropriate correction for tide and sea state conditions,

e) tilt and structure orientation,

f) riser and J-tube contact with the sea floor, and

g) sea floor profile.

NOTE A comprehensive set of photographs, particularly of critical joints, taken while the structure is still in the fabrication yard can be very helpful in planning inspection and interpreting inspection data.

23.6.2 Periodic inspections

23.6.2.1 Detailed scope of work and timing

The timing and detailed scope of work of periodic inspections shall be determined from the inspection strategy (see 23.4) and inspection programme (see 23.5). In the absence of a structural inspection strategy and programme, the default periodic inspection programme given in 23.7 shall be used.

23.6.2.2 Level I periodic inspections

A level I inspection shall consist of a below water verification of the performance of the cathodic protection system (e.g. a drop cell), and of an above-water visual survey, to determine

— the effectiveness of the corrosion protection system,

— deterioration of coating systems,

— excessive corrosion,

— bent, missing, or damaged members, and

— indications of obvious overloading, design deficiencies and any use which is inconsistent with the structure's original purpose.

This inspection shall include a general examination of all structural members in the splash zone and above water, concentrating on the condition of the more critical areas such as topsides legs, girders and trusses. If above water damage is detected, non-destructive testing shall be used when visual inspection cannot fully determine the extent of damage. If the level I inspection indicates that underwater damage is possible, a level II inspection should be conducted as soon as conditions permit.

23.6.2.3 Level II periodic inspections

A level II inspection shall consist of a general underwater visual inspection, to detect the presence of any or all of the following:

— excessive corrosion;

— effects of accidental or environmental overloading;

— scour, sea floor instability;

— damage detectable in a visual swim-round survey;

— design or construction deficiencies;

— presence of debris;

— excessive marine growth.

The inspection shall include the measurement of cathodic potentials of pre-selected critical areas. Detection of significant structural damage during a level II inspection shall initiate a level III inspection. The level III inspection, if required, should be conducted as soon as conditions permit.

23.6.2.4 Level III periodic inspections

A level III inspection shall consist of an underwater close visual inspection (CVI) of pre-selected areas and/or areas of known or suspected damage. Such areas shall be sufficiently cleaned of marine growth to permit thorough inspection. Pre-selection of areas should be based on results of an engineering evaluation of areas particularly susceptible to structural damage and to areas where repeatable inspections are desirable in order to monitor integrity over time. Flooded member detection (FMD) can provide an acceptable alternative to CVI of pre-selected areas where the structural form and sites of potential damage would lead to flooding. Engineering judgment should be used to determine optimum use of FMD and/or close visual inspection of joints. Close visual inspection of pre-selected areas for corrosion monitoring shall be included as part of the level III inspection.

Detection of significant structural damage during a level III inspection shall initiate a level IV inspection where visual inspection alone cannot determine the extent of damage. The level IV inspection, if required, should be conducted as soon as conditions permit.

23.6.2.5 Level IV periodic inspections

A level IV inspection shall consist of underwater NDT of areas pre-selected from results of a level III inspection or from known or suspected damage. A level IV inspection shall include detailed inspection and measurement of damaged areas.

A level III and/or level IV inspection of fatigue sensitive joints and/or areas susceptible to cracking can be necessary to determine if damage has occurred. Monitoring fatigue sensitive joints, and/or reported linear crack-like indications, may be an acceptable alternative to analytical verification.

23.6.3 Special inspections

Special inspections shall be undertaken

a) to assess the performance of repairs undertaken to ensure the fitness-for-purpose of the structure, the minimum requirement for such repairs being a visual inspection (with marine growth cleaning as necessary) conducted approximately 1 year after completion of the repair,

b) to monitor known defects, damage, local corrosion, scour, or other conditions which could potentially affect the fitness-for-purpose of the structure, risers and J-tubes, conductors, or various appurtenances, and

c) as required to inform an assessment (see Clause 24), reuse (see Clause 25) or decommissioning.

23.6.4 Unscheduled inspections

An inspection shall be conducted as soon as practical after the occurrence of an environmental event (e.g. storm, earthquake, mudslide) exceeding that for which the structure was designed or assessed, or of a significant accidental action (e.g. vessel impact, dropped object, explosion). The minimum scope shall include the following:

a) a visual inspection without marine growth cleaning that provides full coverage from sea floor to top of the structure (members and joints), conductors, risers, and various appurtenances, and which includes checking the seabed conditions at the legs/piles and looking for debris and damage;

b) visual confirmation of the existence of sacrificial anodes, electrodes, and any other corrosion protection material/equipment.

Particular attention shall be given to detecting damage and indirect signs of damage, such as areas of missing marine growth.

In the case of accidental actions sufficient inspection and assessment shall be undertaken to establish the total extent of any damage, and any mitigation measures (e.g. repairs and downmanning) required.

23.7 Default periodic inspection requirements

23.7.1 General

In the absence of an in-service structural inspection strategy in accordance with 23.4, the following default inspection requirements (23.7.2 to 23.7.5) shall apply. Inspection requirements less than these requirements can be justified when an inspection strategy is developed and maintained in accordance with 23.4.

Subclauses 23.7.2 to 23.7.5 outline the default requirements for the various inspection types identified in 23.6.

These default requirements address only the concerns of safeguarding human life and protecting the environment. Additional inspection can be needed to meet statutory requirements, owner's corporate policy or industry standards/practices.

Compliance with these default requirements does not guarantee structural reliability or fitness-for-purpose. Specifically, the default requirements do not address underdesign or design errors unless an assessment is performed (see Clause 24) and the assessment results are incorporated into an inspection strategy.

In the absence of a structure-specific SIM, system periodic inspections shall be conducted at the intervals specified in Table 23.7-1.

Table 23.7-1 — Maximum inspection intervals for default periodic inspection programme

Exposure level[a]	Level I inspection[b]	Level II inspection[c]	Level III inspection[c]	Level IV inspection
L1	Annual	3 years	5 years	Determined from level III inspection results
L2	Annual	5 years	10 years	Determined from level III inspection results
L3	Annual	5 years	not required	not required

[a] The exposure levels are defined in 6.6.

[b] The timing of the first periodic level I inspection shall be determined from the date platform installation was completed.

[c] The timing for the first periodic level II and level III inspections shall be determined from the date of the baseline inspection described in 23.6.1.

23.7.2 Default scope for level I periodic inspection

The default scope for a level I periodic inspection shall consist of

a) visual inspection without marine growth cleaning of the above water parts of the structure, and

b) cathodic potential readings of at least one structure leg using a drop cell or other suitable equipment.

23.7.3 Default scope for level II periodic inspection

The default scope for a level II periodic inspection shall consist of

a) visual inspection without marine growth cleaning of the above water parts of the structure,

b) cathodic potential readings of at least one structure leg using a drop cell or other suitable equipment, and

c) a general visual survey of the full structure with particular attention to members, joints, appurtenances, and appurtenance connections.

23.7.4 Default scope for level III periodic inspection

The default scope for a level III periodic inspection shall consist of the following activities.

a) The full scope of a baseline inspection as specified in 23.6.1.

b) Flooded member detection (FMD) of the following components where they are located underwater and were designed to be unflooded:

   — at least 50 % of all primary structural members selected to be representative of the full structure and covering potentially damage/fatigue prone areas such as conductor guide framing in the upper structural bays;

   — key support members for risers, J-tubes, conductors (first underwater framing level only), service caissons and other appurtenances.

   In lieu of the FMD requirements given in b) above, marine growth cleaning and close visual inspection of at least 20 or of 5 % of the total population (whichever is smaller) of primary member end connections, including a minimum of five primary brace to leg connections, may be substituted. Where the structural configuration precludes the use of FMD, close visual inspection (CVI) shall be substituted.

   A level IV periodic inspection, as described in 23.7.5, may also be substituted for the FMD requirement.

c) Marine growth measurements on selected members at a representative set of elevations from mean sea level to the sea floor.

d) For structures with sacrificial anodes: an estimate of the approximate percent depletion of all the anodes, original or retrofit, located on the structure.

e) For structures with impressed current systems: visual survey of the state of the anodes and reference electrodes. Dielectric shields shall also be thoroughly inspected to ensure that they are undamaged, free from discontinuities, and satisfactorily bonded to the structure.

23.7.5 Default scope for level IV periodic inspection

The default scope for a level IV periodic inspection shall consist of the following activities.

a) The full scope of a baseline inspection as specified in 23.6.1.

b) Marine growth cleaning (as required) and detailed inspection of representative welds at nodal joints (member end connections) and other critical locations as determined from the inspection programme. 100 % of each weld length shall be inspected. The degree of marine growth cleaning shall be sufficient to permit thorough inspection.

c) Marine growth measurements on selected members at a representative set of elevations from mean sea level to the sea floor.

d) For structures with sacrificial anodes: an estimate of the approximate depletion of all the anodes, original or retrofit, located on the structure.

e) For structures with impressed current systems: a visual survey of the state of the anodes and reference electrodes. Dielectric shields shall also be thoroughly inspected to ensure that they are undamaged, free from discontinuities, and satisfactorily bonded to the structure.

23.8 Personnel qualifications

23.8.1 Evaluation and inspection strategy

All evaluations (see 23.3) and the development and maintenance of the inspection strategy (see 23.4) shall be performed by engineering practitioners with the following qualifications:

a) familiarity with relevant information about the specific structure(s) under consideration;

b) knowledgeable about underwater corrosion processes and control;

c) competent in offshore structural engineering;

d) experienced in offshore inspection planning;

e) knowledgeable about inspection tools and techniques;

f) cognizant of general inspection findings in the offshore industry.

These engineering practitioners should also be involved in the other phases of the structural integrity management cycle shown in Figure 23.1-1. Further description of each qualification is provided in A.23.8.

23.8.2 Data collection and update

The database for the evaluations shall be developed, populated and updated by engineers and technicians who are

— familiar with the structural arrangement,

— familiar with inspection techniques and inspection recording protocols,

— familiar with analysis techniques and interpretation of analysis results,

— familiar with database administration and requirements for recovery of data, and

— familiar with material specifications, certificates and welding specifications and qualification practices.

There is no requirement for such a database to be computerized; however, data shall be readily recoverable when required.

23.8.3 Inspection programme

Details of the inspection programme scope of work and specifications shall be developed by engineering practitioners who possess

— familiarity with the content of the overall inspection strategy, and

— experience and technical expertise commensurate with inspection tasks to be performed.

Offshore execution of the inspection programme requires diving supervisors and divers (when diving is included in the inspection programme), ROV operators (when ROV operations are included in the inspection programme) and data recorders. All should be qualified for their assigned tasks. They should possess

a) formal qualifications recognized by international or equivalent regional standards, where standards exist for their tasks,

b) knowledge of how and where to look for damage and situations that could lead to damage,

c) training and experience in the methods to be employed,

d) familiarity with the structure owner's data validation, recording and quality requirements.

For divers who will be performing NDT, in addition, accredited training and qualifications or underwater pre-qualification trials are required.

24 Assessment of existing structures

24.1 General

This clause gives procedures for the assessment of existing fixed steel offshore structures to demonstrate their fitness-for-purpose. The aims and procedures are also applicable to topsides structures.

The owner shall maintain and demonstrate the fitness-for-purpose of the structure for its specific site conditions and operational requirements, based on the principles given in this clause.

A structure is deemed fit-for-purpose when the risk of structural failure leading to unacceptable consequences is sufficiently low. The process described in this clause may be followed for the assessment of fitness-for-purpose. The acceptable level of risk depends on regulatory requirements supplemented by regional or industry standards and practice (see Annex H).

A structure that complies with the requirements given in Clauses 6 to 23 of this International Standard may be considered fit-for-purpose.

For structures that do not comply with Clauses 6 to 23, the owner shall seek to reduce the risk associated with a failure, as much as is reasonably practicable, by effecting risk prevention and mitigation measures. In determining the practicability of effecting these measures, the balance between the benefits and the cost, time and difficulty of implementing the measures should be considered. Local conditions and circumstances and the degree of confidence in the data and techniques used in the assessment should also be considered.

Demonstration of adequate fitness-for-purpose may include justified deviation from Clauses 6 to 23, or modifications to either the structure or its operation (i.e. prevention and mitigation measures). Deviations from Clauses 6 to 23 shall be reviewed and approved by the regulator, where one exists, and/or personnel with sufficient knowledge and experience of the behaviour of fixed steel offshore structures to enable them to understand and assess the implications of the deviations. The rationale for the deviations shall be documented.

Prevention and mitigation measures for reducing the occurrence and consequences of structural failure should be considered during all stages of the assessment process. If the structure is strengthened because of a failure to pass the assessment, the design of the strengthening should enable the structure to comply with Clauses 6 to 23.

24.2 Assessment process

Assessment is an integral part of the evaluation phase of the SIM process described in Clause 23 and shown in Figure 24.2-1. An assessment shall be undertaken if any of the initiators specified in 24.4 are triggered, even where a full SIM system is not in place. An assessment shall consider all relevant available data.

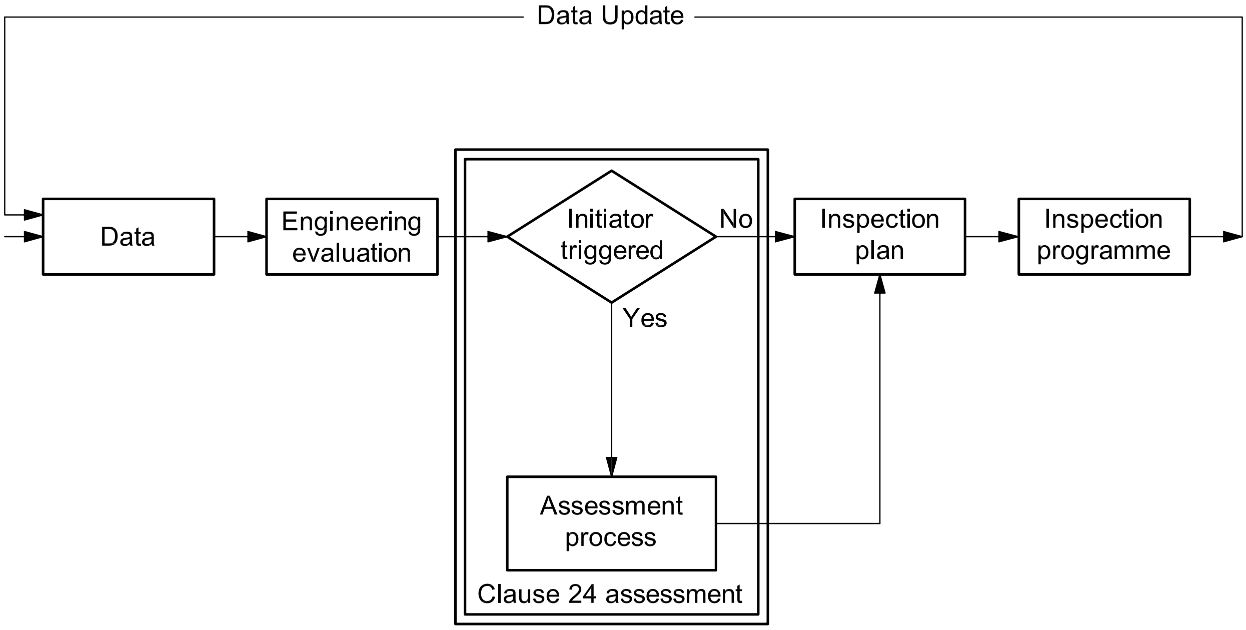


Figure 24.2-1 — Assessment flow chart

For existing structures, it is permissible to accept limited individual component “failure” (i.e. action effects exceeding the component strength as given in this International Standard), provided that both the reserve against overall system failure and deformations remain acceptable. The assessment process involves detailed review, analysis, testing or calculation of the aspects of the design that do not comply with Clauses 6 to 23. The current state of scientific and technical knowledge and the best available data should be used in this process. It is recommended that simple techniques be exhausted prior to the use of technically complex analyses.

Figure 24.2-2 shows a flow chart of the assessment process. As can be seen, there are several points at which an assessment can be completed and it is not always necessary for an assessment to include numerical analysis. Prevention and mitigation measures should be considered at all stages of the assessment process (see 24.10). The design level analysis may be by-passed by going directly to a non-linear ultimate strength analysis (see 24.9). An assessment is completed when satisfactory compliance with respect to the appropriate acceptance criteria is achieved (see 24.5).

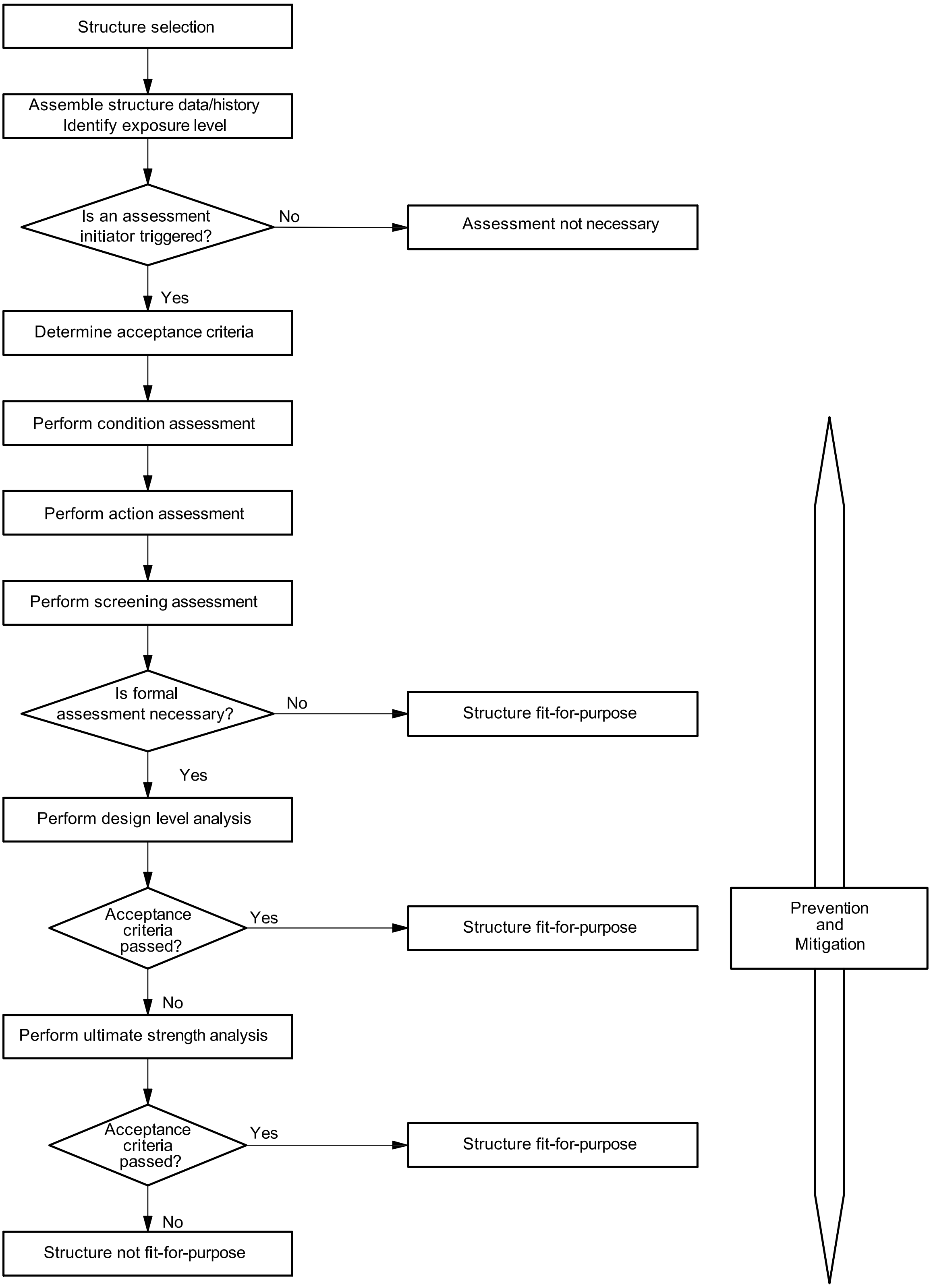


Figure 24.2-2 — Flow chart of the assessment process

The main elements in the process of initiating and conducting an assessment of a structure are the following:

a) assemble data on the structure, its history and exposure level, see 24.3;

b) determine if any assessment initiators are triggered, see 24.4;

c) determine acceptance criteria, see 24.5;

d) assess the condition of the structure, see 24.6;

e) assess the actions, see 24.7;

f) screen the structure in comparison with similar structures, see 24.8;

g) perform a resistance assessment, see 24.9, using

   1) design level analysis,

   2) ultimate strength level analysis, and

   3) prevention and mitigation, see 24.10.

All stages of the assessment shall be fully documented and become part of the structure data.

24.3 Data collection

24.3.1 Structure data and history

The structure data required for assessment are similar to those required for a formal SIM system, see Clause 23. The following data shall, where possible, be reviewed as part of the assessment:

a) general information on structure/configuration;

b) original design information;

c) construction information;

d) information on structure history;

e) information on present condition.

This data shall include results of numerical analyses, engineering evaluations and/or previous assessments.

24.3.2 Exposure level

Acceptance criteria for assessment can depend on the exposure level of the platform. The exposure level shall be determined in accordance with 6.6. The life safety and consequences categories shall be determined for the conditions for which the platform is being assessed, taking into account any changes in use that have been made or that are intended.

24.4 Structural assessment initiators

An existing structure shall be assessed to demonstrate its fitness-for-purpose if one or more of the following conditions exist.

a) Changes from the original design or previous assessment basis, including

   1) addition of personnel or facilities such that the platform exposure level is changed to a more onerous level,

   2) modification to the facilities such that the magnitude or disposition of the permanent, variable or environmental actions on a structure are more onerous,

   3) more onerous environmental conditions and/or criteria,

   4) more onerous component or foundation resistance data and/or criteria,

   5) physical changes to the structure's design basis, e.g. excessive scour or subsidence, and

   6) inadequate deck height, such that waves associated with previous or new criteria will impact the deck, and provided such action was not previously considered.

b) Damage or deterioration of a primary structural component: minor structural damage can be assessed by appropriate local analysis without performing a full assessment; however, cumulative effects of multiple damage shall be documented and included in a full assessment, where appropriate.

c) Exceedance of design service life, if either

   — the fatigue life (including safety factors) is less than the required extended service life, or

   — degradation of the structure due to corrosion is present, or is likely to occur, within the required extended service life.

   An extension of the design service life can be accepted without a full assessment if inspection of the structure shows that time-dependent degradation (i.e. fatigue and corrosion) has not become significant and that there have been no changes to the design criteria [any changes to the original design basis are assessment initiators, see a) above].

A structure which has been totally decommissioned (e.g. an unmanned platform with inactive flowlines and all wells plugged and abandoned), or a structure in the process of being removed (e.g. wells being plugged and abandoned) generally does not need to be subjected to the assessment process.

When an initiator is triggered, the available information shall be evaluated by a competent person who shall

— define any immediate action that is required,

— define the scope of the assessment,

— generate an overall assessment strategy and execution plan,

— define the level of detail and scope of any further information that is required, and

— determine if a full assessment is necessary.

When deciding whether a full assessment is necessary, the following factors shall, as a minimum, be considered:

— the significance of the initiator triggering the assessment with respect to the structural system's reliability;

— the platform's exposure level;

— the recorded historical performance of the structure, including fabrication and installation records.

If, following the evaluation, it is determined that a full assessment is not necessary, the factors and all the pertinent information supporting this decision shall be documented.

24.5 Acceptance criteria

24.5.1 Methods for determining acceptance criteria

Appropriate acceptance criteria shall be developed for the exposure level of the structure being considered in the fitness-for-purpose assessment. Criteria can be defined by statutory requirements, owner's corporate policy, or industry standards/practices, as appropriate. Where criteria have been developed for structures on a regional basis, the criteria are provided in Annex H.

Acceptance criteria may be developed based on the following methods, subject to the limitations noted in A.24.5.1:

a) through the use of explicitly calculated probabilities of failure;

b) through risk based structural reserve strength ratio factors (RSR) developed for location specific and generic structure exposure levels;

c) by comparison with similar platforms, a structure shall not be considered as fit-for-purpose by comparison to a similar structure which itself has been determined to be fit-for-purpose by comparison to another structure;

d) based on prior exposure, e.g. survival of an event that is known with confidence to have been as severe as, or more severe than, the event that would be considered in the actual ultimate system strength analysis.

Acceptance criteria may be developed for different exposure levels in terms of

— reduced actions to be applied in the assessment, e.g. corresponding to shorter return periods, and

— revised resistance criteria, e.g. reduced RSRs

Acceptance criteria shall be applied only for specific regions and on the basis for which they were derived unless it can be demonstrated that the criteria are conservative for the structure being assessed. It should be recognized that adopting reduced acceptance criteria for assessment of existing structures implies a higher probability of failure than acceptable for new structure designs.

24.5.2 Factors for consideration with acceptance criteria

When developing acceptance criteria and when considering the results of an assessment against such criteria, the following factors shall be considered, where appropriate:

a) remaining service life;

b) structural condition monitoring records;

c) long-term loading environment;

d) degree of confidence in modelling assumptions (i.e. uncertainty in actions and resistance, including effects of structural damage and deterioration);

e) sensitivity to analysis assumptions;

f) redundancy and collapse behaviour;

g) structural deformation (with particular consideration to potential escalation events and impairment of lifesaving equipment, escape routes and temporary refuges).

Compliance with acceptance criteria shall not remove the need to consider prevention and mitigation risk reduction measures.

Consideration shall be given to establishing performance criteria against which the adequacy of the screening criteria can be monitored for future assessment.

24.6 Structure condition assessment

24.6.1 General

Sufficient information shall be collected to allow an engineering assessment of the structure's overall structural integrity, which shall include a current inventory of the structure's structural condition and facilities. The owner shall ensure that any assumptions are reasonable and that information gathered is both accurate and representative of current and proposed conditions at the time of the assessment. Particular attention shall be given to data that cannot be explicitly verified, e.g. pile penetrations. Additional details can be found in A.24.6.1.

24.6.2 Topsides surveys

Data relating to the current state of the topsides structure shall be considered. Where data are not available, are ambiguous or thought to be inaccurate, additional walk-around surveys of the topsides structure and facilities shall collect the necessary information.

24.6.3 Underwater and splash zone surveys

In the absence of inspection data, a Level II inspection, as described in 23.6.2.3, as a minimum shall be performed to determine the general condition of the structure. In some instances, additional detailed inspection can be necessary to verify suspected damage, deterioration due to age, absence of joint cans, major modifications, lack of structural drawings, drawings whose accuracy is suspect, poor inspection records, or analytical findings.

24.6.4 Foundation data

Available on-site or near-site soil borings shall be reviewed. Many older structures were installed based on soil boring information at a considerable distance from the installation site. Interpretation of the soil profile can be improved based on more recent site investigations (with improved sampling techniques and in-place tests) performed for other nearby structures. Pile driving records and internal soil plug data may also be used in support of an assessment. Additional details can be found in A.24.6.4.

If more recent and refined geophysical data are available, these may be used to correlate with soil boring data in order to develop an improved foundation model. Any available data from drilling or pipeline surveys shall be considered.

24.7 Actions assessment

24.7.1 General

The assessment of actions shall be performed in accordance with Clause 9 with exceptions, modifications, and additions as noted in 24.7.2 to 24.7.5.

24.7.2 Metocean parameters and environmental actions

The metocean (meteorological and oceanographic) data required for an assessment are the same as for design, as are environmental design situations and actions. In some cases, a reduced return period may be considered for assessment, see 24.5.

24.7.3 Deck elevation and additional environmental actions

The existing deck height, with an allowance for any future subsidence within the design service life, shall be determined. The deck height shall be checked for potential inundation, as this can limit the overall structural reliability. The asymmetry of the wave crest versus wave height should be considered, preferably supported by measured data. If wave inundation of the deck is expected, resistance assessment shall be based on ultimate strength analysis, see 24.9.3.

24.7.4 Seismic design considerations

The seismic design considerations are as given in Clause 11.

24.7.5 Ice conditions and actions due to ice

Guidance on ice conditions and actions due to ice is given in ISO 19901-1 for certain areas.

24.8 Screening assessment

Consideration of the findings from the structure condition (see 24.6) and action (see 24.7) assessments shall be used in conjunction with the acceptance criteria (see 24.5) to determine whether the structure is fit-for-purpose or whether a full resistance assessment (see 24.9) is required. Prior exposure of the structure, results of previous resistance assessments or comparison with similar structures may be used to demonstrate the structure is fit-for-purpose even with the changes which have initiated the assessment process (see 24.4). If the action assessment demonstrates wave contact with the deck is expected, a resistance assessment using the ultimate strength analysis procedure (see 24.9.3) shall be performed and the structure shall not be deemed fit-for-purpose based only on a screening assessment. If wave contact with the deck is not expected and a screening assessment demonstrates that the structure is fit-for-purpose, then the basis shall be fully documented and form part of the structural data within the SIM. If the structure cannot be shown to be fit-for-purpose based on a screening assessment, a full resistance assessment shall be performed (see 24.9).

24.9 Resistance assessment

24.9.1 General

The assessment of structural component resistance shall be performed in accordance with Clauses 12 to 15 and 17 with the exceptions, modifications and additions given below. This procedure may be by-passed by using the non-linear ultimate strength procedure specified in 24.9.3.

A structure shall be evaluated based on its current condition or future intended condition, taking into account any damage, repair, scour, modifications or other factors which can affect its performance or integrity, as given in 24.4.

A structural analysis of the complete structural system, when considered necessary, shall be performed on a three-dimensional model that is of sufficient detail to accurately represent the stiffness of, and actions on, the structure and the foundation. The stiffness and strength of the topsides shall also be adequately modelled. Guidance on structural modelling aspects is given in Clause 12.

Special attention should be given to a rational representation of the actual stiffness of damaged or corroded members and joints.

For structures in areas subjected to low temperatures, special attention should be given to exposed critical connections made of steel not originally specified for low temperature service.

24.9.2 Design level analysis procedure

24.9.2.1 General

Resistance assessment may be performed using the design level analysis outlined in 24.9.2.2 to 24.9.2.4, which is a check of the structure following the same approach as for a new design.

All appropriate partial action and resistance factors given in this International Standard shall be applied in a design level analysis.

24.9.2.2 Structural steel design

24.9.2.2.1 Members

The assessment of structural members shall comply with the requirements of Clause 13; damaged or repaired members shall be evaluated using a rational engineering approach.

Refined member checks may be performed on a case-by-case basis with consideration to the inherent conservative assumptions contained within the design provisions given in Clause 13. Additional information can be found in A.24.9.2.2.

24.9.2.2.2 Connections

The assessment of structural connections shall comply with the requirements of Clauses 14 and 15, with the following exceptions:

— there is no requirement for joint strength to be limited to its brace member strengths, i.e. Equations (14.2.2) and (14.3.11) do not apply to assessment;

— the strength of ungrouted and grouted joints may be based on experimental or analytical studies, provided that these results make due provision for the acceptance criteria for data and for statistical uncertainties underlying the development of the formulae contained in Clauses 14 and 15.

Tubular joints may be assessed for the specific forces derived from the global analysis.

24.9.2.2.3 Fatigue

As part of the assessment process for future service life, consideration shall be given to the accumulated degradation effects due to fatigue and to the future accumulation of fatigue damage during the remaining service life. Durability criteria for structures where fatigue is relevant to the assessment are given in A.24.9.2.2. The inspection history may be used to demonstrate adequate future fatigue durability, giving consideration to both the reliability of inspection results and crack growth behaviour, in lieu of the analytical procedures described in Clause 16. Future inspection requirements shall comply with the principles given in Clause 23.

24.9.2.3 Foundations

Non-linear modelling of piles, as described in Clause 17, shall be used to determine pile capacities. The increase in pile capacities due to ageing effects may be considered, if this can be justified at the specific structure location.

24.9.2.4 Fitness-for-purpose

If all components within the structure and foundation are assessed to have utilizations less than or equal to unity, the structure may be considered to be fit-for-purpose, and no further analysis is required.

For assessment, exceedance of the axial capacity and/or the lateral resistance of an individual pile is acceptable if the total foundation system can be demonstrated to have adequate reserve.

Structures which do not pass the design level check shall have risk prevention and mitigation measures implemented (see 24.10) and/or shall be assessed using either

— a linear elastic redundancy analysis (see A.24.9.2.4;), or

— an ultimate strength analysis (see 24.9.3).

24.9.3 Ultimate strength analysis procedure

24.9.3.1 General

An ultimate strength analysis is intended to demonstrate that a structure has adequate strength and stability to withstand a significant overload, with respect to the actions determined from 24.7. Local overstress and potential local damage are acceptable, but total collapse or excessive/damaging deformations shall be avoided.

The reserve strength ratio (RSR) shall be determined in accordance with 7.10, using the analysis methods described in 12.5 and 12.6 to determine the best estimate of the system strength. The RSR shall be determined for all wave directions and the lowest value obtained shall be the structure's RSR.

In addition to the RSR, consideration shall be given to other relevant factors noted in 24.5.2 that affect the acceptance criteria against which fitness-for-purpose is to be evaluated. The specific method of analysis (e.g. static pushover, cyclic or time domain analysis) depends on the type of extreme environmental action applied to the structure and the intended purpose of the analysis.

24.9.3.2 Wave-in-deck actions

Special attention shall be given to modelling of the deck if wave inundation is expected, see 24.7. The following aspects shall be considered.

a) The increase in overturning moment and base shear, due to the inundation. The increase in overturning moment can be associated with a relatively small increment in base shear and can lead to a significant increase in actions on the foundation piles.

b) In regions where the wave height increases significantly with increased return periods, explicitly including wave/current actions on the deck within an ultimate strength analysis.

All structures shall demonstrate adequate strength and stability. If the deck is inundated, the additional environmental action shall be considered. A method for calculating this additional action is given in A.24.7.3, which may be used in lieu of more advanced techniques.

The provisions of 24.9.2.2.3 shall apply even if the design level analysis is by-passed.

24.9.3.3 Component modelling

The following points apply to non-linear ultimate strength analysis.

a) The mean strength of undamaged members, joints and piles may be established by using mean strength values in the formulae given in Clauses 13 to 15 and 17 to give an unbiased estimate of strength.

b) The ultimate strength, and post ultimate behaviour of damaged or repaired components of the structure may be evaluated using a rational engineering approach based on improved procedures and knowledge from testing.

c) The mean expected yield strength may be used instead of the specified minimum yield strength of the material. Actual coupon tests from the material may also be used. However, adequate account of the increased statistical variability of small samples should be made in the evaluation of the estimate of ultimate system strength. Strain rate effects beyond the normal (fast) mill tension test shall not be used.

d) Studies and tests have indicated that effective length (K) factors are substantially lower for components of a frame subjected to overload than those specified in 13.5. Lower values may be used, if it can be demonstrated that they are both applicable and substantiated.

24.9.3.4 Fitness-for-purpose

If the minimum RSR value (R_{RS}) calculated from the ultimate strength analysis meets or exceeds the acceptance criteria from 24.5.1, the structure may be considered to be fit-for-purpose, and no further analysis is required.

In the absence of specific acceptance criteria, fitness-for-purpose shall be assessed against the RSR value required for a new structure with the same exposure level and in the same location, see 7.10.

24.10 Prevention and mitigation

Prevention and mitigation of risk using appropriate measures should be considered at all levels of assessment for structures that do not comply with Clauses 6 to 23. These measures should consider all aspects of the platform operations.

Possible prevention measures include structural strengthening and reduction of actions on the structure. Mitigation includes such measures as de-manning and hydrocarbon inventory reduction that will reduce the consequences of a failure of the structure.

Consideration of prevention and mitigation measures should involve all engineering disciplines, in order to ensure comprehensive consideration of any prevention and mitigation measures.

25 Structure reuse

25.1 General

In general, structures are designed for onshore fabrication, loadout, transportation and offshore installation, as well as for the in-place situations. By reversing the construction sequence, structures can be removed, back-loaded, transported, modified (if required) and then reinstalled at new sites.

Any or all parts of a platform may be reused, depending on their suitability and condition, and it is not necessary to reuse a structure with the same topsides or *vice-versa*. Components may be renewed or replaced as appropriate; in particular, the majority of piles are likely to require major refurbishment or replacement.

Structures that are reused shall comply with this clause as well as with the other design clauses of this International Standard. This clause contains additional requirements for structures to be reused with respect to fatigue, materials, inspection, removal, and reinstallation.

25.2 Fatigue considerations for reused structures

Fatigue sensitive locations in reused structures, especially tubular joints, shall be inspected in accordance with 25.4. The calculation of fatigue damage shall include appropriate allowances for fatigue damage accumulated during the prior in-service period(s) and all transportation phases, in addition to the design service life at the new location. For structures from, or to be reused in, areas of the world where fatigue can be a governing design criterion, the sum of the previously accumulated fatigue damage, D_1, and the future fatigue damage, D_2, for the intended reuse period shall not exceed a value of 1,0, including appropriate fatigue damage design factors for both periods (see 16.12).

25.3 Steel in reused structures

The type and grade of steel used in primary structure components of reused structures shall be determined from the original records. Reused structures with tubular connections in which heavy-wall tubular joints have been fabricated from toughness class CV1 (or better) steel shall be inspected in accordance with the requirements of 25.4, including UT inspection to detect the occurrence of unacceptable defects. Where

Clause 19 requires the use of toughness class CV2Z or better steel, steel inspection (including UT inspection) shall be used to confirm the absence of unacceptable defects.

The chemical composition and mechanical properties of all materials shall be verified for consistency with the assumptions made for the design analysis at the new location. Particular attention shall be paid to confirming the identification and properties of any steel of group II or higher.

Mill certificates or other documentation from the original fabrication with adequate material traceability may be used. If material certificates are unavailable, or if there is any doubt about the correlation of certificates with the locations of steel within the structure, specimens shall be taken from the structure and tested by a certified laboratory to confirm both chemical and mechanical properties.

25.4 Inspection of structures to be reused

25.4.1 General

When structures are considered for reuse, adequate inspection and testing shall be undertaken to verify suitability for the intended application. Care shall be taken in evaluating the inspection results if the structure is inspected prior to its removal from the original location; in such cases the limitations of *in situ* inspection shall be taken into account, both in its ability to provide a thorough inspection and the possibility of further damage during removal and transportation.

Inspection programmes prepared for evaluation of existing structures being considered for reuse shall be sufficiently detailed to establish the condition of the structures as described in Clause 23. Additionally, inspection should be performed to verify the absence of any damage which can impair the structure's ability to withstand actions imposed during all phases of removal operations from the prior location.

All design assumptions shall be verified by inspection, including material composition and properties (see 25.3), connection integrity and extent of any corrosion or other degradation due to prior service.

Assessment of the condition of used structures should begin with a review of existing documentation from the original construction of the structure, together with results of any in-service surveys. Any evidence of damage or repairs, for which adequate investigation and assessment has not previously been undertaken, shall be reviewed and assessed in accordance with Clauses 23 and 24. Such damage can occur from environmental overload, ship collisions and operational activities. Validation of any repair systems and their integrity should be conducted.

25.4.2 Initial condition assessment of structural members and connections

The extent, quality, timing and findings of NDT performed during the original fabrication and during periodic in-service inspections of the structure shall be reviewed. Where adequate documentation exists, and the extent of inspection and weld qualities are consistent with current acceptance criteria, inspection may be limited to an investigation of in-service damage due to overload or fatigue.

Where adequate NDT documentation is not available, an initial spot survey of the structure should be made to provide guidance for the assessment and to assist in the formulation of a detailed inspection plan. The spot survey should include a general visual inspection of the whole structure, to detect any gross structural damage (parted connections, missing members, dented or buckled members, corrosion damage, etc.). Structural members and connections having in-service damage shall be 100 % inspected, using appropriate NDT techniques.

25.4.3 Extent of weld inspection

The extent of weld inspection undertaken on a structure to be reused should be determined from assessments of the utilizations of members and joints in the structure in both prior service and future reuse. The inspection of welds at joints, particularly where they are fatigue-sensitive, should be directed to the higher stressed areas where fatigue cracks are more likely to initiate.

Welds should be sufficiently cleaned for effective NDT. NDT inspection methods and deployment of inspection equipment are described in A.23. Guidance on the extent of inspection for various existing parts of the structure is given in A.25.4.3.

Weld inspection may take place either with the structure still at the previous location, or with the structure removed from the water, as appropriate, but the requirements for the extent of the inspection shall be the same.

Inspection of all new components and connections shall be in accordance with Clauses 20 and 21. Inspection of existing welds should generally comply with the requirements of Clause 23.

25.4.4 Corrosion protection systems

The integrity of corrosion protection systems should be verified in accordance with Clauses 18 and 23. Verification should include assessment of remaining anode materials, anode connections, impressed current functionality and the condition of any protective coatings (e.g. splash zone coatings and wraps), and possible hidden damage under coatings.

25.4.5 Inspection for removal of structures from prior location

The inspection required for reuse may be undertaken before or after removal of the structure from the previous location (see 25.4.3). All inspection required for the safe removal of the structure shall be determined during the planning stages (see 25.5) and executed prior to the commencement of the removal. As part of this process, the structure and equipment weights shall be verified.

25.5 Removal and reinstallation

Removal and reinstallation shall follow all relevant requirements of Clause 22.

An offshore construction plan shall be prepared for structure removal and reinstallation. This plan shall include the methods and procedures for the safe execution of the entire removal and reuse operation, including the following specific activities and considerations:

— removal of the topsides, equipment, appurtenances, conductors, piling and structure;

— loading of these items onto the transportation barges, including the crane barge if appropriate;

— seafastening and transportation of the topsides, equipment, appurtenances, conductors, piling and structure to the new location or to and from an onshore location used for storage and/or refurbishment;

— any necessary modifications or refurbishment of topsides, equipment, appurtenances, conductors, piling and structure;

— reinstallation of the reused or replaced structure, piling, appurtenances, topsides, equipment and conductors.

Lift weights and centres of gravity should be defined in the procedures and checked during lifting.

Particular emphasis should be placed on the prevention of damage from the removal operations of any structure components intended for reuse.

25.6 In-service inspection and structural integrity management

A new inspection and structural integrity management (SIM) strategy shall be developed for the structure at the new location, taking full account of the structure's history, design service life and the requirements of Clause 23.

Annex A
(informative)

Additional information and guidance

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

A.1 Scope

No guidance is offered.

A.2 Normative references

No guidance is offered.

A.3 Terms and definitions

No guidance is offered.

A.4 Symbols

No guidance is offered.

A.5 Abbreviated terms

No guidance is offered.

A.6 Overall considerations

A.6.1 Types of fixed steel offshore structure

No guidance is offered.

A.6.2 Planning

No guidance is offered.

A.6.3 Service and operational considerations

A.6.3.1 General considerations

If the structure is to be set over an existing well with the wellhead above water, information is needed on the dimensions of the tree, size of conductor pipe, and the elevations of the casing head flange and top of wellhead above mean low water. If the existing well is a temporary subsea completion, plans should be made for locating the well and setting the structure such that the well can later be extended above the surface of the water.

A.6.3.2 Water depth

No guidance is offered.

A.6.3.3 Structural configuration

A.6.3.3.1 General

No guidance is offered.

A.6.3.3.2 Deck elevation

A safety margin or air gap is required between the crest of the design wave and the lowest point (beam, equipment or fixing) of the lowest deck of the platform such that abnormal wave crests do not impinge on the deck. This is necessary, since very large actions can occur if a wave hits the deck. If there is insufficient deck elevation, wave impact can determine the reliability of the structure. Where possible, deck height should be chosen so that the frequency of wave impact on the deck is compatible with the target failure rate of the structure. Structural reliability arguments indicate that air gaps selected in accordance with traditional procedures should be increased or reduced, depending on the region in which the platform is located. The air gap further ensures that green water does not interfere with platform operation or safety devices.

Any 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. The deck elevation can be set by either of the following methods:

a) a rational process, using long-term surface elevation statistics and reliability considerations, going to various levels of complexity; or

b) experience and judgment, if a rational approach is not possible.

Information on air gap requirements for certain geographical areas is given in Annex H.

Method a) above can be followed if the metocean database is sufficiently accurate and comprehensive. Account may be taken of the joint probability of tide, surge height, and crest heights to estimate the maximum surface elevation relative to the deck. In this case, a probability of non-exceedance close to the target failure rate of the structure may be used with no additional air gap allowance added.

When method a) above is not appropriate, the deck elevation, h, above the mean sea level can be estimated from Equation (A.6.3-1) or Equation (A.6.3-2).

If storm surge is not expected to occur at the same time as the abnormal wave crest, Equation (A.6.3-1) applies:

$$h = \sqrt{a^2 + s^2 + t^2} + f \tag{A.6.3-1}$$

If storm surge is expected to occur at the same time as the abnormal wave crest, Equation (A.6.3-2) applies:

$$h = \sqrt{(a + s)^2 + t^2} + f \tag{A.6.3-2}$$

where

a is the abnormal wave crest height;

s is the extreme storm surge;

t is the maximum elevation of the tide relative to the mean sea level;

f is the expected sum of subsidence, settlement and sea level rise over the design service life of the structure.

For deep and intermediate water depths a can be approximated to

$$a > 1{,}3\, a_{100} \tag{A.6.3-3}$$

$$a > a_{100} + 1{,}5 \text{ m} \tag{A.6.3-4}$$

where a_{100} is the extreme wave crest height with a return period of 100 years.

The estimate for h obtained with this procedure is indicative and suitable for conceptual design studies. The owner should review the deck elevation prior to detailed design.

In general, no platform processing elements, piping, or equipment should be located below the lower deck in the designated air gap. However, when it is unavoidable to position such items as minor sub-cellars, sumps, drains, or production piping in the air gap, provisions should be made for the actions due to waves developed on these items.

NOTE 1,5 m is the traditional value used for air gap but analysis of metocean data has shown that it does not always allow sufficient reliability in certain geographical areas.

A.6.3.3.3 Equipment and material layouts

No guidance is offered.

A.6.3.4 Access and auxiliary systems

No guidance is offered.

A.6.4 Safety considerations

No guidance is offered.

A.6.5 Environmental considerations

A.6.5.1 General

No guidance is offered.

A.6.5.2 Selecting design metocean parameters and action factors

Worldwide experience of drilling and production structures supports the use of 100 year return metocean parameters. The partial action and resistance factors recommended herein are intended for use in designing the structure in accordance with this practice.

Where the recommended partial action factors are not used, the values used should be based on a risk and reliability analysis. This analysis should include the estimated cost of the structure designed to resist actions from environmental conditions using various action factors, the probability of damage or loss when subjected to environmental conditions of various recurrence intervals, the financial loss due to damage or loss including lost production, clean-up, structure and well replacement, etc. The reliability approach used to develop the partial action and resistance factors is reasonable for such a study.

A.6.6 Exposure levels

A.6.6.1 General

Guidance in the following subclauses relates to life-safety and the consequences of platform loss. Life-safety concerns personnel belonging to the normal complement of personnel on the platform. *Consequences* is a broad notion, which includes factors such as the life-safety of personnel that are not part of the normal complement of the platform (e.g. rescue personnel or those who might be brought in to clean up and make safe a platform after an incident), damage to the environment and anticipated economic losses (see 6.6.3).

The emphasis is normally on the first and second of these factors. Owners may, however, choose to consider economic factors more specifically in setting exposure levels to give a higher level of reliability of a structure.

NOTE When considering economic losses, in addition to loss of the platform and associated equipment, and damage to connecting pipelines, the loss of reserves can be considered if the field is subsequently abandoned. Removal costs include the salvage of the collapsed structure, re-entering and plugging damaged wells, and cleanup of the sea floor at the site. If the site is not to be abandoned, restoration costs include replacing the structure and equipment and re-entering the wells. Other costs include repair, re-routing or reconnecting pipelines to the new structure.

A.6.6.2 Life-safety categories

When determining the length of time required for evacuation of a category S2 manned evacuated platform, consideration should be given to

a) the number of personnel to be evacuated,

b) the distances involved,

c) the capacity and operating limitations of the evacuating equipment,

d) the type and size of docking / landings, refuelling, egress facilities on the platform, and

e) the environmental conditions anticipated to occur throughout the evacuation effort.

An occasionally manned platform (e.g. one manned for only short duration activities such as maintenance, construction, workover operations, drilling and decommissioning) may be classified as S3, unmanned, provided that the activities are scheduled such that all exposure of personnel to any design environmental event is both minimal and as low as reasonably practicable.

A.6.6.3 Consequence categories

The degree to which negative consequences could result from platform collapse is a judgment which should be based on the importance of the structure to the safety of any personnel associated with the platform on a normal basis or in relation to the failure, the potential damage to the environment, the owner’s overall operation and the level of economic losses that could be sustained as a result of the collapse.

When considering pollution and environmental damage and the cost of mitigation, particular attention should be given to the hydrocarbons contained in the topsides process inventory, possible leakage of damaged wells or pipelines, and the proximity of the platform to the shoreline or to environmentally sensitive areas such as coral reefs, estuaries, and wildlife refuges. The potential amount of liquid hydrocarbons or sour gas released from these sources should be considerably less than the available inventory from each source. The factors affecting the release from each source are discussed below.

— **Topsides inventory**

  At the time of a platform collapse, liquid hydrocarbon in the vessels and piping is not likely to be suddenly released. Due to the continuing integrity of most of the vessels, piping and valves, it is most likely that very little of the inventory will be released. Thus, it is judged that significant liquid hydrocarbon release is a concern only in those cases where the topsides inventory includes large capacity containment vessels.

— **Wells**

  The liquid hydrocarbon or sour gas release from wells depends on several variables. The primary variable is the reliability of the subsurface safety valves (SSSVs), which are fail-safe closed or otherwise activated when an abnormal flow situation is sensed. Where regulations require the use and maintenance of SSSVs, it can be judged that uncontrolled flow from wells is not a concern for the platform assessment. Where SSSVs are not used and the wells can freely flow (e.g. under well pressure), the flow from wells can be a significant concern.

Any liquid hydrocarbon or sour gas above the SSSV could be lost over time in a manner similar to a ruptured pipeline; however, the quantity will be small and does not necessarily have a significant impact.

— **Pipelines**

The potential for liquid hydrocarbon or sour gas release from pipelines or risers is a major concern because of the many possible causes of rupture (e.g. platform collapse, soil bottom movement, intolerable unsupported span lengths, and anchor snag). Only platform collapse is addressed in this International Standard. Platform collapse is likely to rupture pipelines or risers near to or within the structure. For the design environmental event where the lines are not flowing, the maximum liquid hydrocarbon or sour gas release is likely to be substantially less than the inventory of the line. The amount of product released will depend on several variables such as the line size, the residual pressure in the line, the gas content of the liquid hydrocarbon, the undulations of the line along its route, and other secondary parameters.

Of significant concern are major oil transport lines which are long, large in diameter and have a large inventory. In-field lines, which are smaller and have less inventory, are not necessarily a concern.

A.6.6.4 Determination of exposure level

No guidance is offered.

A.6.7 Assessment of existing structures

No guidance is offered.

A.6.8 Structure reuse

No guidance is offered.

A.7 General design requirements

A.7.1 General

No guidance is offered.

A.7.2 Incorporating limit states

No guidance is offered.

A.7.3 Determining design situations

No guidance is offered.

A.7.4 Structural modelling and analysis

In general, partial action factors are applied to external actions before calculating internal forces. If the structure and support conditions are linear, it does not matter whether the partial action factor is applied to the external action or to the internal force. However, when non-linearity exists, applying the partial action factors to the external actions ensures compatible deformations and equilibrium of internal forces at a load level closer to an overload condition.

A.7.5 Design for pre-service and removal situations

No guidance is offered.

A.7.6 Design for the in-place situation

Except for fatigue, the design requirements for exposure level L1 are the same for structures of all design service lives. For example, the return period and action factor for wind, wave and current actions are the same whether the structure has a design service life of 5 years or 30 years. This ensures that all structures have the same reliability in any single year. This is desirable for the safety of personnel on the structure and to control the exposure to potential consequences of failure.

A shorter design service life reduces the probability of encountering an extreme environmental event and so increases the lifetime probability of survival. The owner of exposure levels L2 and L3 structures may take advantage of this fact. However any change in the return period and design action factors impacting the risk to life, injury, damage to the environment or economic losses to parties other than the owner should be determined by a reliability analysis. Changes impacting solely on economic losses to the owner are at the discretion of the owner.

A.7.7 Determination of resistances

A.7.7.1 General

In many cases it is possible to derive the resistance of a complex component from a computer simulation, usually using finite element techniques. Guidance on the use of such techniques, including the selection of appropriate elements, is given in Clause 12.

There is a significant possibility of errors being introduced in both physical tests and in computer simulations. Considerations for physical tests are given in A.7.7.2. When using computer simulations, potential errors in deriving a representative resistance should be taken into account by

— calibrating the results of analyses to similar results from tests or design formulae;

— using computer simulations where the statistical variation is introduced in the derivation of the representative resistance value based on “known” values of the variance.

Where no test data are sufficiently similar, requirements are given in 7.7.5.

A.7.7.2 Physical testing to derive resistances

Large scale testing is a valid method of determining the behaviour and the resistance of structures and of components of structures; however, such tests can be expensive and can also introduce additional unknowns. The factors that should be considered in designing tests include

— the expected mode of failure,

— the scale of the test pieces and consequent scaling factors required,

— the methods of application of the loads (e.g. by the use of hydraulic rams),

— the arrangement and stiffness of the support and reaction framing,

— the method of controlling the application of the load (e.g. by controlling the applied load, or by controlling the applied displacements),

— whether the method of load application and control will affect the test conclusions,

— other actions on the real structure that are not replicated in the test (e.g. hydrostatic pressure),

— the use of different materials to represent the materials intended for the real component,

— the methods of monitoring the behaviour of the test piece including strain and displacement gauges, and

— the determination of the resistance of the component from the test results, such as the load/displacement relationships.

The test arrangement should be designed to minimize the influence or take account of these considerations. The experience of the test contractors can be of great benefit in optimizing the test programme and in understanding the results of testing.

Where the resistance is derived by physical testing, and where the variability of the behaviour is expected to follow a normal distribution, Equation (A.7.7-1) may be used to derive the design resistance R_d:

$$R_d = \frac{\eta_d}{\gamma_R} \cdot R_{mean} \left(1 - k_n \cdot V_R\right) \tag{A.7.7-1}$$

where

η_d is a correction factor;

γ_R is the partial resistance factor according to this International Standard;

R_{mean} is the mean of the test results;

k_n is a statistical factor;

V_R is the coefficient of variation.

Factor η_d is intended to account for the differences between the test arrangements and the real component, such as scaling, differences in support and restraint between test piece and real conditions, differences in yield strengths and strain rates, etc. Consequently, the value of η_d is specific to the arrangement of both the real component and the test arrangement and specific guidance cannot be given in this International Standard.

The partial factor approach adopted in this International Standard takes account of the statistical variations in the actions applied to a structure and in the resistances of the components of the structure. In deriving the representative resistances formulated in Clauses 13 to 15, large numbers of tests have been undertaken and the mean strengths and coefficients of variation are well understood. Where few tests are available the natural coefficient of variation, V_R, is estimated from the sample using Equation (A.7.7-2):

$$V_R = \frac{1}{R_{mean}} \sqrt{\frac{1}{n-1} \sum \left(x_i - R_{mean}\right)^2} \tag{A.7.7-2}$$

where

R_{mean} is the mean of the test results;

x_i are individual test results;

n is the number of tests.

The factor, k_n, used in Equation (A.7.7-1) is a statistically derived factor for taking account of the low number of test results that could be available from the test programme. Where no data from similar testing can be found for deriving an appropriate coefficient of variation, a minimum of three tests should be undertaken. Less conservative values of k_n may be used if comparisons with data from similar tests can be used. Factor k_n is given in Table A.7.7-1 for cases where V_R can be established from similar data (V_R known), and where similar data are not available (V_R unknown). Table A.7.7-1 assumes a normal distribution and is for a 95 % exceedance at the 50 % confidence level.

Table A.7.7-1 — Statistical factor, k_n[A.7.7-1]

n	1	2	3	4	5	6	8	10	20	30	∞
V_R **known**	2,31	2,01	1,89	1,83	1,80	1,77	1,74	1,72	1,68	1,67	1,64
V_R **unknown**	—	—	3,37	2,63	2,33	2,18	2,00	1,92	1,76	1,73	1,64

Where similar tests are available the coefficient of variation may be taken from those tests and used in Equation (A.7.7-1). The degree of similarity of tests is a matter of judgment, but this approach should only be used if no additional behavioural effects, such as a change in mode of failure, are likely. Cases where appropriate data are available and where the coefficient of variation from those data is used in Equation (A.7.7-1) are classed as "V_R known" in Table A.7.7-1.

It is not always necessary to conduct a series of tests on nominally similar specimens to establish k_n and V_R if the mode of failure is well-defined. In such cases, it is sometimes possible to replicate only that part of the specimen which causes the failure. For example, if it is known that failure of a particular bolted connection is always associated with bolt shear, then simpler tests on bolts in shear may be used to establish V_R and k_n. The mean R_{mean} should, however, be established with reference to at least three complete specimens. Care needs to be exercised to ensure that all relevant parts of the complete component which can affect the failure (including stiffness) are included in any simpler tests.

A.7.7.3 Resistances derived from computer simulations validated by physical testing

A methodology for deriving resistances from computer simulations validated by physical testing is presented below. In some cases it will be possible to use data from existing tests rather than requiring new tests. This methodology essentially captures both any bias in the predictions and the natural variability of the structural behaviour, and is particularly appropriate when there are a series of components with varying parameters, such that the test series can cover the parameter ranges. A series of tests should be simulated and the ratios of measured resistances (R_m) to predicted resistances (R_p) calculated. The predicted resistances may be based on the measured yield strengths of the test pieces. A representative ratio $(R_m/R_p)_{rep}$ is calculated using Equation (A.7.7-3):

$$(R_m/R_p)_{rep} = (R_m/R_p)_{mean}\,(1 - k_n\,V_{m/p}) \qquad \text{(A.7.7-3)}$$

where

$(R_m/R_p)_{mean}$ is the mean ratio of measured to predicted strengths;

k_n is a statistical factor based on the number of tests, see A.7.7.2;

$V_{m/p}$ is the coefficient of variation of the ratio R_m/R_p.

The design resistance of the *in situ* component, R_d, is given by Equation (A.7.7-4):

$$R_d = \frac{1}{\gamma_R} \cdot (R_m / R_p)_{rep} \cdot R_{p,s} \qquad \text{(A.7.7-4)}$$

where

γ_R is the partial resistance factor from this International Standard;

$(R_m/R_p)_{rep}$ is the representative ratio from Equation (A.7.7-3);

$R_{p,s}$ is the predicted resistance of the *in situ* component from the simulation, based on its specified minimum yield strength.

In order to use Equation (A.7.7-4), the analyst should be satisfied that the variability of the behaviour of the *in situ* component is no greater than the variability of the test specimens; this requires sufficient tests to be undertaken to evaluate the variability to the appropriate level of confidence. Consideration should be given to the effects of any additional parameters such as welding stresses and geometrical imperfections.

The value of $R_{p,s}$ in Equation (A.7.7-4) is the predicted resistance of the component based on computer simulation and taking into account the accuracy of computer simulations of available test configurations. As the analysis introduces additional parameters (e.g. mesh size selection) the same techniques should be used for analyses of both the real component and the available test configurations.

A.7.7.4 Resistances derived from computer simulations validated against design formulae

When validating computer simulations against design formulae it should be noted that the formulae from Clauses 13 to 15 give representative resistances, and therefore take account of structural behaviour variability. The design resistance, R_d, is given by Equation (A.7.7-5):

$$R_d = \frac{1}{\gamma_R} \cdot (R_f / R_p)_{mean} \cdot R_{p,s} \tag{A.7.7-5}$$

where

γ_R is the partial resistance factor from this International Standard;

$(R_f / R_p)_{mean}$ is the mean ratio of design formulae resistances (R_f) to simulation resistances (R_p) for the same values of parameters;

$R_{p,s}$ is the predicted strength of the real component from the simulation.

Both R_p and $R_{p,s}$ should be calculated using the specified minimum yield strength.

The value of $R_{p,s}$ in Equation (A.7.7-5) is the predicted resistance of the component based on the computer simulation and taking into account the accuracy of computer simulations of arrangements covered by design formulations. As the analysis introduces additional parameters (e.g. mesh size selection) the same techniques should be used for analyses of both the real component and the simulations of the arrangements covered by design formulae. Depending on the accuracy of the simulations, it may be appropriate to adjust modelled material properties or to apply a factor to the results of the analysis in determining $R_{p,s}$.

In order to use Equation (A.7.7-5), the analyst should be satisfied that the arrangement of the component is sufficiently similar to that from which the design formula was derived and that both will have the same mode of failure. An example of an appropriate use of Equation (A.7.7-5) is to extrapolate just beyond the limits of validity of available design formulae.

A.7.7.5 Resistances derived from unvalidated computer simulations

It is preferable that computer simulations be validated against true structural performance; 7.7.3 directly achieves this, while 7.7.4 achieves validation through design formulae which in themselves are derived from test data. It is, however, recognized that there are occasions in which physical testing is unrealistic, for example, due to scale and complexity, and hence this guidance is given for deriving resistances from computer simulations for such cases. Where no physical testing or design formulae validation is available, R_d may be derived from Equation (A.7.7-6):

$$R_d = \frac{\eta_a}{\gamma_R} \cdot R_{p,rep} \tag{A.7.7-6}$$

where

η_a is a modelling conversion factor;

γ_R is the partial resistance factor from this International Standard;

$R_{p,rep}$ is the representative resistance derived from the analysis;

$$R_{p,rep} = R_{p,s}(1 - k_1 V_R) \tag{A.7.7-7}$$

where

$R_{p,s}$ is the resistance derived from the simulation;

k_1 is a statistical factor equal to 2,31;

V_R is a coefficient of variation to reflect the variability of structural behaviour.

Since the value of k_1 is quite onerous, multiple analyses may be performed to allow the use of k_n in place of k_1, with $R_{p,s}$ becoming the mean resistance from the analyses. Where this is done, and to reduce some of the systematic errors being introduced, the multiple analyses should be undertaken using different analysis techniques and different analysts.

A similar approach to that in A.7.7.2 is used for the determination of the design value from computer simulations, with a modelling factor, η_a, provided to take account of differences between the simulation and the real conditions. Modelling factor η_a will approach a value of 1,0 as the real conditions are more closely simulated; it should be determined by the analyst and is not intended to be used as an indiscriminate reduction factor to enhance safety.

Each of the analysis techniques should be validated by comparison to available test data, and the value of $R_{p,s}$ used in Equation (A.7.7-7) should be the mean of the resistances from the various analyses. The appropriate value of V_R depends on the expected mode of failure. The analyst should consider any available data on testing and select the value of V_R accordingly; however, V_R should not be less than the values given in Table A.7.7-2.

Table A.7.7-2 — Minimum coefficients of variation

Type of failure expected	Minimum coefficient of variation, V_R
Excessive yielding or gross deformation	0,05
Local buckling	0,11
Overall buckling	0,17

It is customary to use the specified minimum yield strength (SMYS) as the yield strength when determining $R_{p,s}$. The use of SMYS together with the use of V_R from Table A.7.7-2 can, however, lead to over-conservatism. Where the analyst has sufficient understanding of the statistical variation of the material yield strength, $f_{y,n}$, the value of yield strength, f_y, used in the analysis may be modified. In the absence of data on the material yield strength the analysis may be based on a yield strength from Equation (A.7.7-8):

$$f_{y,a} = \frac{f_{y,n}}{(1 - 1{,}64 \times 0{,}05)} = 1{,}09\, f_{y,n} \tag{A.7.7-8}$$

where

$f_{y,a}$ is the yield strength used in the analysis when determining $R_{p,s}$;

$f_{y,n}$ is the characteristic yield strength for a normal distribution that is not truncated by removing rejected samples with yield strengths below SMYS ($f_{y,n}$ is normally slightly larger than SMYS).

The values of 1,64 and 0,05 are valid for a large sample size and a typical coefficient of variation for structural steel yield strength.

It should be noted that steel for offshore structures is usually purchased by stipulating the SMYS, and that, while steel with yield strengths falling below the SMYS will be rejected, it cannot be assumed that the yield strength distribution of purchased steel is normal. For this reason, $f_{y,a}$ should not be taken as greater than the mean of the SMYS of the steel being used and the SMYS of the next grade of steel available.

The value of $k_1 = 2{,}31$ for use in Equation (A.7.7-7) is the value appropriate for a single physical test from Table A.7.7-1. As the solution of a particular computer simulation will always produce the same result, there is a requirement to use different analysis techniques and different analysts if the opportunity to use a lower value of k_n is desired. The variations between the analyses should include as many of the following as possible:

— different analysts, with little collaboration in establishing their simulations;

— different analysis programs;

— different formulations for the behaviour of the finite elements;

— different mesh sizes and arrangements.

Where computer simulations are used to extend or explore the range of a parameter, which is difficult to control in physical tests (e.g. imperfections in columns) then it is possible to use the results to establish V_R if reasonable correspondence to the existing results is obtained with relevant simulations.

A.7.8 Strength and stability checks

These requirements refer to the partial factor design format. In general, different components (tubular members, joints, piles, etc.) have different partial resistance factors (also known as strength reduction factors or ϕ factors), while different actions (self weight, variable actions due to movable equipment, environmental actions, etc.) have different partial action factors (also known as γ factors). The partial action factors may further vary depending on which other actions are combined with them.

A.7.9 Robustness

The robustness concept is closely related to accidental actions, consequences of human error, and failure of equipment. In ISO 19900 these situations are denoted “hazardous circumstances” or “hazards”. Robustness is also important in the event of serious but unidentified fatigue damage.

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

ISO 19900 requires the following approach:

— careful planning of all phases of development and operation;

— avoidance of the structural effects of the hazards by either eliminating the source or by bypassing and overcoming them;

— minimizing the consequences, or

— designing for hazards.

When the hazard cannot reliably be avoided, the designer has a choice between minimizing the consequences (i.e. the consequences of losing a structural component due to a hazard), or designing for the hazard (i.e. making the component strong enough to resist the hazard). In the first case, the structure should be designed such that all structural components that can be exposed to hazards are non-critical. In the second case, critical components that can be exposed to hazards are made strong enough to resist the hazards considered.

It should be noted that robustness requirements do not imply that structures can survive removal of any structural component. If there is no hazard, then there is no requirement in relation to robustness. Also, only one hazard at the time should be considered.

A.7.10 Reserve strength

A.7.10.1 New structures

The global strength of structural systems able to safely withstand environmental overload situations is greater than that inferred from component-based design. This is due to different sources of reserve strength in the design such as

- conservatism in code design equations,
- the difference between actual and representative material yield strengths (SMYS),
- additional strength required by load cases other than in-place environmental actions,
- a designer's choice to oversize members in order to comply with standard sizes, reduce the number of different sizes used, facilitate fabrication or speed up the design process, and
- reserve strength offered by system behaviour.

A non-linear pushover analysis can be used to estimate system reserve strength in terms of the RSR, and thus quantify the structure's ability to resist environmental overload.

The partial action factors for environmental actions specified in Clause 9 are derived from a database of space frame structures, which assumes a certain minimum degree of reserve strength due to various sources such as listed above.

The determination of system reserve strength is not required for design of new space frame structures. For structures with less inherent reserve system strength than that of a typical space frame type structure, an adequate safety level can be achieved by determining the structure's RSR and adjusting $\gamma_{f,E}$ so as to yield the required system reserve strength.

In lieu of more refined methods, this can be done following ISO 19900 to determine the partial action factor, $\gamma_{f,Em}$, for the extreme environmental action:

$$\gamma_{f,Em} = \frac{R_{RS,Typ}}{R_{RS,M}} \gamma_{f,E} \tag{A.7.10-1}$$

where

$R_{RS,Typ}$ is the minimum acceptable value of RSR for a conventionally framed structure for the relevant exposure level;

$R_{RS,M}$ is the RSR for the structural arrangement being considered, found either from a pushover analysis or from lower bound estimates from component resistances;

$\gamma_{f,E}$ is the partial action factor for extreme environmental actions (see 9.9 and A.9.9).

$R_{RS,Typ}$ should be close to 1,85 for an L1 structure. This value was calculated in References [A.7.10-1] and [A.7.10-2] from an analysis of North Sea type structures. As the value of $R_{RS,M}$ depends on the component sizes, which are themselves dependent on $\gamma_{f,Em}$, an iterative process can be required to determine both $\gamma_{f,Em}$ and $R_{RS,M}$. Lower values of RSR can be appropriate for L2 and L3 structures.

A.7.10.2 Existing structures

Assessment of existing structures may be based on different levels of refinement. It is recommended that simple techniques be exhausted prior to the use of technically complex analyses.

For existing structural systems non-linear pushover analysis is an option for the designer to document if a structure is fit-for-purpose. The overall safety level of the structure is measured by an estimate of its RSR for environmental actions. Each exposure level is associated with a required minimum reserve strength. Given a certain exposure level, the same RSR requirements apply to all structural configurations (jackets, towers, monotowers, free-standing caissons and braced caissons).

A.7.11 Indirect actions

In general, the ultimate strengths of ductile structural systems are not sensitive to indirect actions, because the behaviour of ductile components beyond full plastic loading is not affected by the initial level of internal stress. Nevertheless, indirect actions can be particularly important in the following instances:

a) where a component loses resistance during buckling (e.g. a thin curved shell or a torsionally buckling nominally straight stiffener);

b) where the indirect stresses move the component's response from the linear region to a near buckling geometrically non-linear region (as a guide, this effect can be important if the indirect action effect is greater than about 20 % of the elastic critical buckling resistance);

c) where the indirect stresses result in significant triaxial tensile stresses, such as the result of weld shrinkage in a complex component (a triaxial stress state can reduce the fracture toughness of the affected steel).

The effects of indirect actions should therefore be checked on a case-by-case basis. Where a) above applies, the indirect actions should be included in the ultimate limit state check; where b) or c) applies, the indirect actions need not be included in the ultimate limit state check, but the enhanced stress ranges should be included in fatigue calculations.

Where indirect actions are considered in detail, they should be included with permanent and variable actions as detailed in Table A.7.11-1 (see 9.2).

Table A.7.11-1 — Inclusion of indirect actions in permanent and variable actions

Indirect action	Included in
Prestressing	Permanent action, G_1
Creep/relaxation	
Settlement	
Permanent temperature loads (air, sea water)	Permanent action, G_2
Production oriented temperature loads	Variable action, Q_1

A.7.12 Structural reliability analysis

No guidance is offered.

A.8 Actions for pre-service and removal situations

A.8.1 General

Actions and corresponding safety checks for pre-service and removal situations are often ignored in standards because their possible consequences are viewed as owner risks and do not affect the general public. For offshore structures, failures during this phase are primarily economic and do not normally involve operating personnel or environmental hazards. The situation changes, however, if structures are partially damaged during pre-service or removal situations and this fact escapes detection. Subsequent capacity to perform functions or resist environmental hazards can then be reduced. Consequently, this International Standard considers such actions as well.

During lifting operations and during the installation stage, the structure is in a state of dynamic equilibrium such that arbitrary use of partial action factors can lead to incorrect and even unsafe results. Unlike other situations, the representative values to be used for lifting and installation analyses, including stiffness parameters and external actions, should be best estimates rather than conservative values. Safety considerations should in these cases be taken into account by performing appropriate sensitivity analyses, instead of applying partial action and resistance factors.

Partial action factors should not be confused with dynamic or impact factors. The latter are the increase in the effect of actions due to inertia. Partial action factors, on the other hand, account for the uncertainty in estimating the effect of individual static and/or dynamic actions.

A.8.2 General requirements

A.8.2.1 Design situations

No guidance is offered.

A.8.2.2 Weight control

Cases where the behaviour of the structure and the magnitude and distribution of the forces in the structure can be very sensitive to the position of the centre of gravity include

— floating situations,

— uprighting,

— lifts with a very eccentric location of the centre of gravity, and

— cases where the arrangement of equipment items leads to high uncertainty in the centre of gravity location.

In normal cases, where the effect of a shift in the centre of gravity is mainly local, this effect can be assumed to be included in the local factor of 8.3.5.

A.8.2.3 Dynamic effects

No guidance is offered.

A.8.2.4 Internal forces

No guidance is offered.

A.8.3 Actions associated with lifting

A.8.3.1 General

Lifting loads depend on the nature of the object being lifted (size, weight, stiffness, etc.), the particulars of the lifting equipment (crane stiffness, hook speed, sling arrangement, etc.), and on the conditions and procedures under which the lift is made (e.g. onshore, sheltered inshore or exposed offshore). The general guidance on lifting loads provided in this International Standard assumes typical lifts in reasonably controlled conditions.

Critical offshore lifts, such as those involving heavy loads approaching the capacity of the lifting equipment or those using dual crane (two cranes on one vessel) or tandem lift (two crane vessels) operations, should be subject to special investigation. This should investigate the dynamic lift forces in cranes, rigging, lifting points and in the lifted structure, as well as the actions due to impact during lift-off and setting down of the load, and should account for the limiting weather conditions in which the operation may proceed. Frequency domain and/or time domain techniques may be used to analyse the relative motions of the lift vessel, the transportation barge and the structure, in order to determine the magnitude of these dynamically induced forces.

Specific lift criteria used for final design should be determined in conjunction with the installation contractor.

For the lifting of an object, the following considerations should be taken into account.

a) Weight and weight growth.

b) Dynamic amplification due to lift-off from a cargo barge and due to motions of the crane vessel.

c) Effects of a shift in the centre of gravity.

d) Tilt, which, for a single crane lift with four slings, is caused by the fact that the centre of gravity is not located exactly under the crane hook, and for a dual crane lift, by the fact that the centre of gravity position is not exactly below the line through both crane hooks and by possible uneven hoisting speeds.

e) Yaw, an effect present only in dual crane lift operations, caused by yawing of the lifted object.

f) Sling force distribution, which depends on sling stiffness, the stiffness of the lifted object, sling length and the hook assembly.

Actions due to lifting on lifting attachments and on other components of the structure should include both vertical and horizontal components, the latter occurring when lift slings are other than vertical. Vertical actions on the lifted object should include the effect of buoyancy, as well as actions imposed by the lifting equipment.

When suspended, the structure will occupy a position such that its centre of gravity is below the centroid of all upward actions on it and such that the structure is in static equilibrium. The attitude of the lift in this state of static equilibrium should be used to determine internal forces in the structure and in the slings. The movement of the lift as it is picked up and set down should be taken into account in determining critical combinations of vertical and horizontal actions at all points, including those to which lifting slings are attached. For lifts where either the crane or the structure to be lifted is on a floating vessel, the selection of the nominal lifting forces should consider the influence of vessel motions.

The lift should be designed such that all structural steel members are proportioned for factored resistances as specified in 7.7. In addition, all critical structural connections and primary members should be designed to have adequate reserve strength to ensure structural integrity during lifting.

A.8.3.2 Dynamic effects

The lifting DAFs are in accordance with certification society rules such as Lloyds[A.8.3-1] and will be addressed in ISO 19901-6[3].

A.8.3.3 Effect of tolerances

The effect of tolerances in a lift analysis of a standard four-point lift may be taken into account by one of the following methods:

— an analysis with one pair of opposite slings assumed to carry 75 % and the other pair 25 % of the hook force, and *vice versa*;

— an analysis with modified sling lengths, e.g. two diagonally opposite slings with increased length, each by an amount corresponding to the total tolerance, for each diagonal in turn.

The analyses described above are very detailed, and yet reality is more complex. For instance, during design, the sling lengths, final location of the centre of gravity, hook geometry and effect of hook assembly hinges, and the stiffness of the slings are not exactly known. The minimum sling force increase given in 8.3.3 compensates for these unknowns.

A.8.3.4 Dual lift

No guidance is offered.

A.8.3.5 Local factor

The objective of the local factors is to account for the sensitivity of framing members to tolerances in the fabrication of lifting attachments and for tolerances in the load transfer from the structure through the lifting attachments to the slings. As the whole weight of the structure normally passes through four lifting attachments, which are essentially pinned connections, the effect of misalignment can be significant. The effect of misalignment, however, reduces quickly away from the lifting attachment, thereby allowing $\gamma_{f,lf}$ to be reduced for members not framing directly into lifting attachments.

A.8.3.6 Member and joint strengths

No guidance is offered.

A.8.3.7 Lifting attachments

No guidance is offered.

A.8.3.8 Slings, shackles and fittings

No guidance is offered.

A.8.4 Actions associated with fabrication

No guidance is offered.

A.8.5 Actions associated with loadout

A.8.5.1 Direct lift

No guidance is offered.

A.8.5.2 Horizontal movement onto barge

No guidance is offered.

A.8.5.3 Self-floating structures

Self-floating structures can be launched from the fabrication yard to float with their own buoyancy for tow to the installation site. The last portion of such a structure leaving the launchways can be subjected to localized forces as the first portion of the structure to enter the water gains buoyancy and causes the structure to rotate from the slope of the ways. Actions should be evaluated for the full travel of the structure down the ways.

A.8.6 Actions associated with transportation

A.8.6.1 General

No guidance is offered.

A.8.6.2 Environmental conditions

The environmental conditions used in determining the motions of the towing arrangement should be established by the owner (in consultation with the marine warranty surveyor if appropriate), taking account of the expected tow route and season. For long ocean tows where the structure and barge are unmanned, the extreme environmental conditions are typically selected to have a probability of exceedance during the tow duration in the range of 1 % to 10 %. The specific value will depend on an evaluation of acceptable risks and consequences. For short duration tows, the environmental conditions should generally have a return period of not less than 1 year for the season in which the tow takes place.

A.8.6.3 Determination of actions

No guidance is offered.

A.8.6.4 Other considerations

No guidance is offered.

A.8.7 Actions associated with installation

A.8.7.1 Lifted structures

No guidance is offered.

A.8.7.2 Launched structures

Barge-launched structures are usually launched at or near the intended destination. The structure is generally moved along slipways, which terminate in rocker arms, on the deck of the barge; these rocker arms distribute reactions into strong points of the structure. A barge can have both primary and secondary rocker arms. As the centre of gravity of the structure moves over the edge of the barge, the structure starts to rotate, causing the rocker arms at the end of the slipways to rotate while the structure continues to slide from the rocker arms. Reactions supporting the structure on the slipways should be evaluated for the full travel of the structure. Deflection of the rocker beam and the effect on internal forces throughout the structure should also be considered. In general, the most severe forces will occur as rotation starts.

A.8.7.3 Crane assisted uprighting of structures

No guidance is offered.

A.8.7.4 Submergence pressures

No guidance is offered.

A.8.7.5 Member flooding

No guidance is offered.

A.8.7.6 Actions on the foundation during installation

A.8.7.6.1 General

The wind, wave, and current conditions used to determine the environmental actions should be established for the installation season, and are typically selected to have a probability of exceedance during the exposure period in the range of 1 % to 10 %. The specific value will depend on an evaluation of acceptable risks and consequences. The exposure period used should take account of the installation sequence and the structure configuration for the various phases, considering relevant operations, such as buoyancy tank removal, pile stabbing, driving and securing, conductor installation, and deck placement. For short exposure durations, the seasonal environmental conditions should generally have a return period of not less than 1 year, unless restrictions are imposed in installation procedures to limit the conditions in which operations may proceed.

A.8.7.6.2 Determination of actions

No guidance is offered.

A.8.8 Actions associated with removal

Examples of actions associated with removal are those due to explosive cutting, the sudden transfer of pile weight to structure and mudmats, pile and mudmat suction forces, lifting forces, reduced buoyancy and increased weight (e.g. from marine growth or grout) compared with those at installation, and concentrated actions during landing the structure on a barge.

A.9 Actions for in-place situations

A.9.1 General

The partial action factors provided in Clause 9 are intended to cover variations in the intensity of direct actions from the specified representative values and as far as appropriate the uncertainties in predicting internal forces.

The values have been primarily calibrated to conditions experienced in the Gulf of Mexico, accounting for the analysis procedures normally followed in applying API RP2A-LRFD[A.9.1-1].

In no instance should partial action factors be used as a substitute for rational analysis, e.g. to account for dynamics.

Partial action factors may be obtained from this International Standard, including any regional information in Annex H, or from national application documents, or where sufficient information is available by an appropriate reliability analysis. Such a reliability analysis includes

a) uncertainty in the intensity of direct actions,

a) probability of simultaneous occurrence of independent actions,

b) accuracy of the prediction of action effects (e.g. internal force), and

c) interpretation of relevant field experience, verification, and control.

Acceptable reliability should be based on failure consequences and compatibility with minimum safety levels intended to be achieved by this International Standard; see 7.10.

A.9.2 Permanent actions (G) and variable actions (Q)

A.9.2.1 Permanent action 1, G_1

No guidance is offered.

A.9.2.2 Permanent action 2, G_2

This category has been separated from traditional variable actions because there is little variability in its magnitude when the equipment or package is known.

A.9.2.3 Variable action 1, Q_1

No guidance is offered.

A.9.2.4 Variable action 2, Q_2

No guidance is offered.

A.9.2.5 Unintentional flooding

As noted in 9.2.1, buoyancy (and hydrostatic pressure) is considered to be a permanent action. This is consistent with the relatively high certainty of the displaced volume of a component.

One uncertainty in the action due to buoyancy is whether the component remains internally dry. The occurrence of unintentional flooding is not well-documented, so rather than developing an *ad hoc* partial action factor, several additional checks are called for. The check of unflooded members as flooded can be done for all members at once, rather than each singly. For intentionally flooded members, the need to ensure positive flooding can be critical to prevent hydrostatic collapse, particularly during rapid submergence that can occur during installation.

The requirement for flooding some fraction of all unflooded members can be modelled by a set of equivalent actions on joints.

A.9.2.6 Position and range of permanent and variable actions

No guidance is offered.

A.9.2.7 Carry down factors

No guidance is offered.

A.9.2.8 Representation of actions from topsides

No guidance is offered.

A.9.2.9 Weight control

No guidance is offered.

A.9.3 Extreme environmental action due to wind, waves and current

No guidance is offered.

A.9.4 Extreme quasi-static action due to wind, waves and current (E_e)

A.9.4.1 Procedure for determining E_e

The return period in years means the inverse of the annual probability of exceedance of a parameter (e.g. a wave height or wind speed). In this context, a 100 year storm, for example, has no meaning; the quantity of interest should be qualified in terms of metocean parameters.

The three methods noted in 9.4.1 are all used to determine the extreme direct action, E_e; further discussion of these methods is given in ISO 19901-1. Method a) in 9.4.1 (100 year return period wave with other associated parameters estimated from correlations) has been used for Gulf of Mexico designs, while method b) (100 year return period wave with 100 year return period wind and 100 year return period current) has been used in the North Sea and many other areas, whereas method c) (100 year return period action or action effect) is a more recent development, suitable when a database of joint occurrences of wind, waves and current is available.

Additional considerations should be given to obtaining the extreme direct action, E_e, for locations where there are strong currents that are not driven by local storms. Such currents can be driven by tides or by deep water currents, such as the Loop Current in the Gulf of Mexico or the Gulf Stream. In this case, method 9.4.1 a) can be acceptable if the storm generated conditions are the predominant contributors to the extreme global environmental action (action effect) and if the appropriate “associated” value of tidal and circulation current can be determined. However, method c) is conceptually more straightforward and preferable. Method b) is the simplest method and ensures an adequate design environmental action (action effect) that can be very conservative compared to the true 100 year return period global environmental action (action effect).

For some areas, substantial databases are becoming available with which it is possible to establish statistics of joint occurrence of wind, wave and current magnitudes and directions. When such a database is available, this should be used to develop environmental conditions based on method 9.4.1 c), which provides the 100 year return period extreme global environmental action on the structure. The corresponding partial action factors and RSR to be used in conjunction with the 100 year return period global environmental action (action effect) should be determined using structural reliability analysis principles, in order to ensure that an appropriate safety level is achieved. This approach provides more consistent reliability (safety) for different geographic areas than has been achieved by the practice of using separate (marginal) statistics of winds, currents, and waves.

A.9.4.2 Direction of extreme wind, waves and current

No guidance is offered.

A.9.4.3 Extreme global actions

No guidance is offered.

A.9.4.4 Extreme local actions and action effects

No guidance is offered.

A.9.4.5 Vortex induced vibrations (VIV)

No guidance is offered.

A.9.5 Extreme quasi-static action caused by waves only (E_{we}) or by waves and currents (E_{wce})

A.9.5.1 Procedure for determining E_{we} and E_{wce}

The procedure depicted in Figure 9.5-1 can be used with any combination of wave and current, not just the 100 year wave and associated current. However, the wave-current combination used as input to the procedure should account for wave-current joint probability insofar as possible. The procedure is intended to account for the principal hydrodynamic phenomena as realistically as is practical within the framework of the calculation of a deterministic quasi-static global environmental action, and has been validated using measured hydrodynamic actions on full-scale structures in simultaneously occurring waves and currents[A.9.5-1]. Therefore, use of wave-current combinations that do not account for wave-current joint probability will result in hydrodynamic actions that are biased high.

When appropriate, the procedure is repeated for various relevant and foreseeable combinations of extreme wave height, wave period, current and wind speeds.

A.9.5.2 Models for hydrodynamic actions

A.9.5.2.1 Morison equation

The use of the local acceleration rather than the total (local plus convective) acceleration in the inertia term of Morison's equation is the subject of ongoing debate. There have been several publications on this topic, see References [A.9.5-2], [A.9.5-3], [A.9.5-4] and [A.9.5-5]. These publications all conclude that the total acceleration should be used. However, it should be noted that these publications all unrealistically assume that the flow does not separate from the cylinder. Realistically, except for very small amplitudes of oscillation (Keulegan-Carpenter number, $K < 3$, see A.9.5.2.3.4), the flow separates on the downstream side of the cylinder, creating a wake of reduced velocity. The local change in velocity across the cylinder due to the convective acceleration in the undisturbed far-field flow is generally much less than the change in velocity due to local flow separation, as implied in Reference [A.9.5-6]. The convective acceleration can also be nearly in phase with the locally incident flow velocity, which leads the undisturbed far-field velocity in oscillatory flow because of “wake encounter”[A.9.5-7]. Therefore, it can be argued that the convective acceleration should be neglected, either because it is small relative to local velocity gradients due to flow separation or because it is already implicitly included in drag coefficients derived from measurements of local actions in separated flow. As a practical matter, the convective acceleration only exceeds 15 % of the local acceleration in steep waves, for which inertia action is generally much smaller than drag action[A.9.5-5].

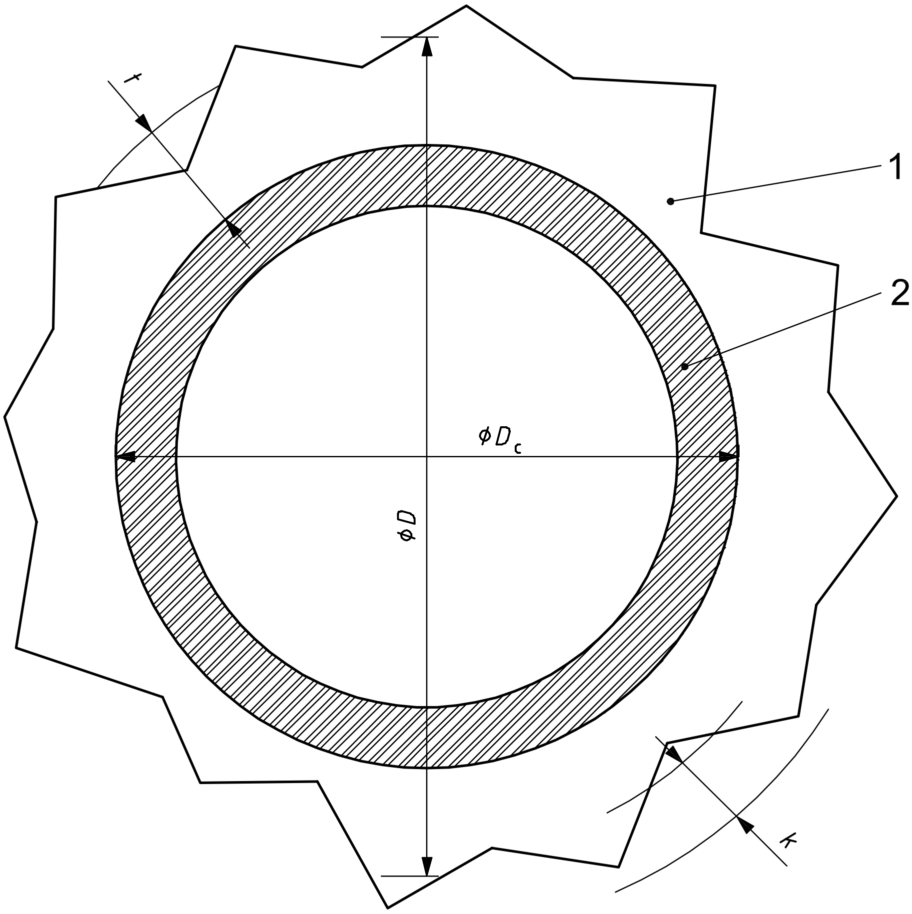
Only the components of velocity and acceleration normal to the member axis are used in computing drag and inertia actions, based on the “flow-independence” or “cross-flow” principle. This principle has been verified in steady sub-critical flow in Reference [A.9.5-8] and in steady post-critical flow in Reference [A.9.5-9]. The data given in Reference [A.9.5-10], as re-interpreted by Reference [A.9.5-11], have shown the flow-independence principle to be acceptable also for inertia actions in one-dimensional oscillatory flow. Therefore, it is reasonable to assume that the flow-independence principle is valid in general for both steady and multidimensional oscillatory flows, with the exception of flows near the unstable, critical Reynolds number regime.

A.9.5.2.2 Marine growth

It is necessary to account for the foreseen influence of marine growth on hydrodynamic action that the structure can experience during its design service life. This influence arises from an increased drag coefficient due to roughness, increased diameter, and addition to density. It will vary over depth and with time throughout the design service life of the structure.

All components of the structure (members, conductors, risers, appurtenances, etc.) are increased in cross-sectional area by marine growth, see Figure A.9.5-1. The effective component diameter (or cross-sectional width for non-circular members or prisms) is $D = D_\mathrm{c} + 2\,t$, where D_c is the “clean” outer diameter and t is the average marine growth thickness that would be obtained by circumferential measurements with a 25 mm to 100 mm wide tape. An additional parameter that affects the drag coefficient of elements with circular cross-sections is the relative roughness, $e = k/D$, where k is the average peak-to-valley height of “hard” growth organisms. Marine growth thickness and roughness are illustrated in Figure A.9.5-1 for a circular cylinder. Marine organisms generally colonize a structure soon after installation. They grow rapidly at the beginning, but growth tapers off after a few years. In some areas, the initial colonization is by one species (e.g. mussels), which is subsequently removed by a predatory species (e.g. starfish), and ultimately replaced by another (e.g. sea anemones). Marine growth has been measured on structures in many areas but should be estimated for other areas.

The thickness of marine growth depends on location. Experience in one area of the world cannot necessarily be applied in another. Where possible, site-specific studies should be conducted to establish marine growth thickness and its dependence on depth below water.



Key

1 hard growth

2 member

t average marine growth thickness

k average marine growth peak to valley height

D effective component diameter, $D = D_c + 2\,t$

D_c diameter of clean member

e relative roughness, $e = k/D$

Figure A.9.5-1 — Definition of surface roughness height and thickness

A.9.5.2.3 Drag and inertia coefficients

A.9.5.2.3.1 General

In the ocean environment, hydrodynamic action predicted by Morison's equation is only an engineering approximation. Morison's equation can match measured drag and inertia actions reasonably well in any particular half wave cycle with the constants, C_d and C_m, but the best-fit values of C_d and C_m vary from one half wave cycle to another. Most of the variation in C_d and C_m can be taken into account by expressing C_d and C_m as functions of

— relative surface roughness $e = k/D$

— Reynolds number $Re = U_m\,D/\upsilon$

— Keulegan-Carpenter number $K = U_m\,T/D$

— current/wave velocity ratio $r = U_c/U_{mo}$

where

k is the average roughness height;

D is the effective diameter (including marine growth);

U_m is the maximum speed (including current) normal to the cylinder axis in a wave cycle;

υ is the kinematic viscosity of water;

T is the wave period;

U_c is the current speed measured in-line with the waves;

U_{mo} is the maximum wave-induced orbital velocity.

A.9.5.2.3.2 Surface roughness

The dependence of the steady-flow drag coefficient at post-critical Reynolds numbers (C_{ds}) on relative surface roughness is shown in Figure A.9.5-2 for “hard” roughness elements. All the data in this figure have been adjusted, where necessary, to account for wind tunnel blockage and the drag coefficient has been referenced to the effective diameter, D, including the roughness elements.

Natural marine growth on structures will generally have a relative surface roughness, $e > 10^{-3}$. Thus, in the absence of better information on the expected value of surface roughness and its variation with depth for a particular site, it is reasonable to assume C_{ds} in the range 1,00 to 1,10 for all members below high tide level. It is still necessary to estimate the thickness of marine growth that will ultimately accumulate in order to estimate D. For elements above high tide level, a reasonable estimate of surface roughness is $k = 0{,}05$ mm, which will give C_{ds} in the range of 0,6 to 0,7 for typical diameters.

All the data in Figure A.9.5-2 are for cylinders that are densely covered with surface roughness elements. Load measurements, see References [A.9.5-12] and [A.9.5-13], show that there is little degradation in the effectiveness of surface roughness for surface coverage as sparse as 10 %, but that roughness effects are negligible for surface coverage less than 3 %.

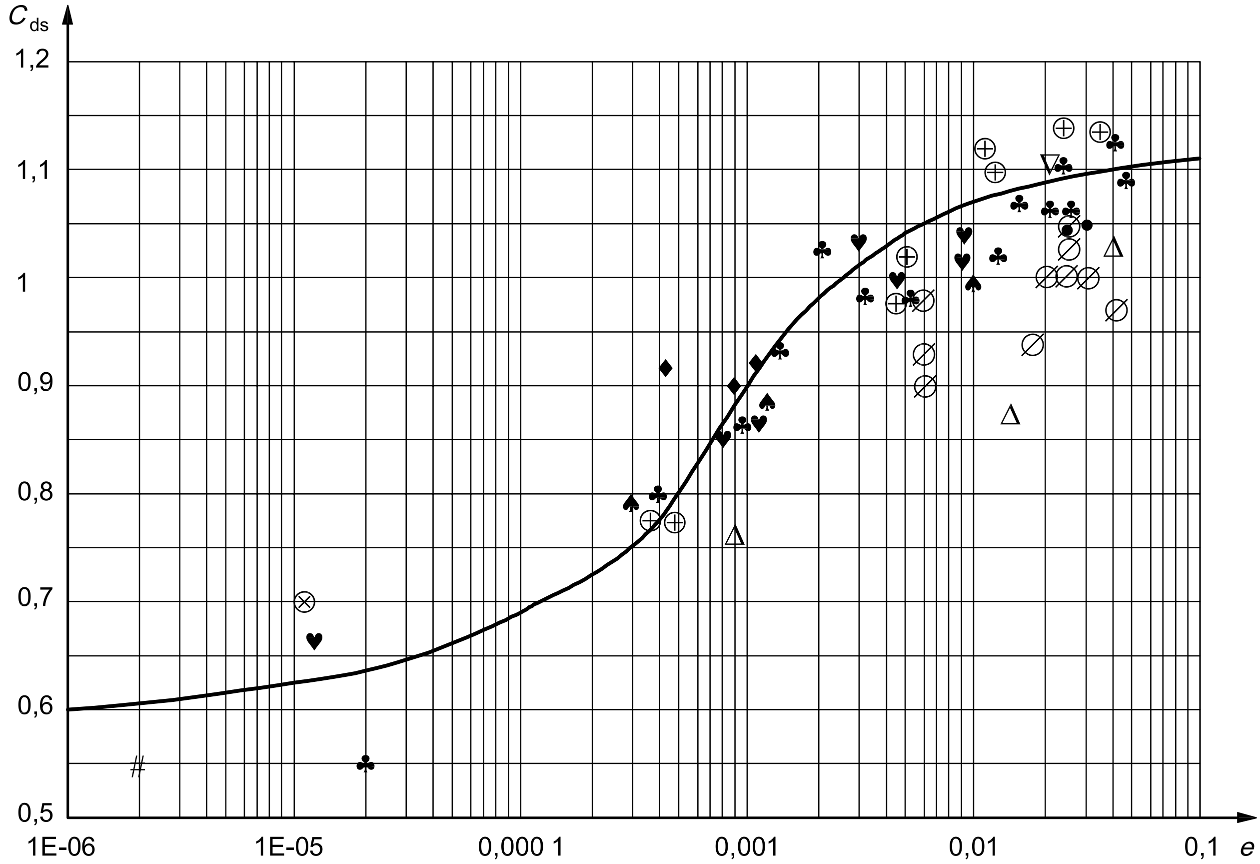
The effect of soft, flexible growth on C_{ds} is poorly understood. Tests reported in Reference [A.9.5-14] indicate that

— soft, fuzzy growth has little effect, as C_{ds} is predominantly determined by the underlying hard growth, and

— anemones and kelp produce drag coefficients similar to those for hard growth.

For cylindrical elements whose cross-section is not circular, C_{ds} may be assumed to be independent of surface roughness. Suitable values are provided in Reference [A.9.5-15].

Surface roughness also affects the inertia coefficient in oscillatory flow. Generally, as C_d increases with roughness, C_m decreases. More information is provided in subsequent discussions.



Key

e relative roughness

C_{ds} steady-flow drag coefficient at post-critical Reynolds numbers

\# Jones(1989)

Δ Blumberg (1961)

• Wolfram (1985)

♣ Miller (1976)

♦ Szechenyl (1975)

♥ Achenbach (1971, 1981)

♠ Wang (1986)

⊗ Roshco (1961)

⊕ Norton (1983)

∅ Nath (1987)

∇ Rodenbusch (1983)

Figure A.9.5-2 — Dependence of steady flow drag coefficient on relative surface roughness

A.9.5.2.3.3 Reynolds number

The hydrodynamic coefficients for elements whose cross-sections have sharp edges are practically independent of the Reynolds number. However, circular cylinders have coefficients that depend on the Reynolds number.

Fortunately, for most offshore structures in the extreme design environment, Reynolds numbers are well into the post-critical flow regime, where C_{ds} for circular cylinders is independent of the Reynolds number. However, in less severe environments, such as considered in fatigue calculations, some platform members can drop into the critical flow regime. Use of the post-critical C_{ds} in these cases would be conservative for static calculations of wave actions but non-conservative for calculating damping of dynamically excited structures.

In laboratory tests of scale models of structures with circular cylindrical members, one should be fully aware of the dependence of C_{ds} on the Reynolds number. In particular, the scale of the model and the surface roughness should be chosen to eliminate or minimize Reynolds number dependence, while the difference between model-scale and full-scale C_{ds} should be considered in the application of model test results to full-scale structures. Further guidance on the dependence of circular cylinder C_{ds} on the Reynolds number can be found in References [A.9.5-16], [A.9.5-8] and [A.9.5-5].

A.9.5.2.3.4 Keulegan-Carpenter number

The Keulegan-Carpenter number, K, is a measure of the unsteadiness of the flow. It is proportional to the distance normal to the member axis travelled by an undisturbed fluid particle in a half wave cycle, normalized by the member diameter. For a typical full-scale space frame structure in design storm conditions, K is generally greater than 40 for members in the "wave zone", and drag action is predominant over inertia action. However, for large diameter legs that can be found on self-floating structures, K can be less than 10, and inertia action is predominant over drag action.

The parameter K is also a measure of the importance of "wake encounter" for nearly vertical (within 15° of vertical) members in waves. As the fluid moves across a member, a wake is created. When oscillatory flow reverses, fluid particles in the wake return sooner and impact the member with greater velocity than undisturbed fluid particles. For larger K, the wake travels farther and decays more before returning to the cylinder and, furthermore, is less likely to strike the cylinder at all if the waves are multidirectional or there is a component of current normal to the principal wave direction. For very large K, wake encounter may be neglected. For smaller K, wake encounter amplifies the drag action for nearly vertical members above its quasi-steady value estimated from undisturbed fluid velocities.

Figure A.9.5-3 shows data for the drag coefficient C_d that are most appropriate for calculating hydrodynamic actions on nearly vertical elements in extreme storm environments. All these data were obtained in the post-critical flow regime, in which C_{ds} is practically independent of the Reynolds number. All data account for wave spreading, i.e. all have two components of motion normal to the element axis. All except the data from "figure of 8" orbits implicitly account for random wave motion. The field data also naturally include an axial component of motion and, to some extent, a steady current. For $K > 12$, with K normalized by C_{ds}, the data for smooth and rough cylinders are reasonably well represented by a single curve in Figure A.9.5-3, as suggested by the far-field, quasi-steady wake model described in Reference [A.9.5-17].

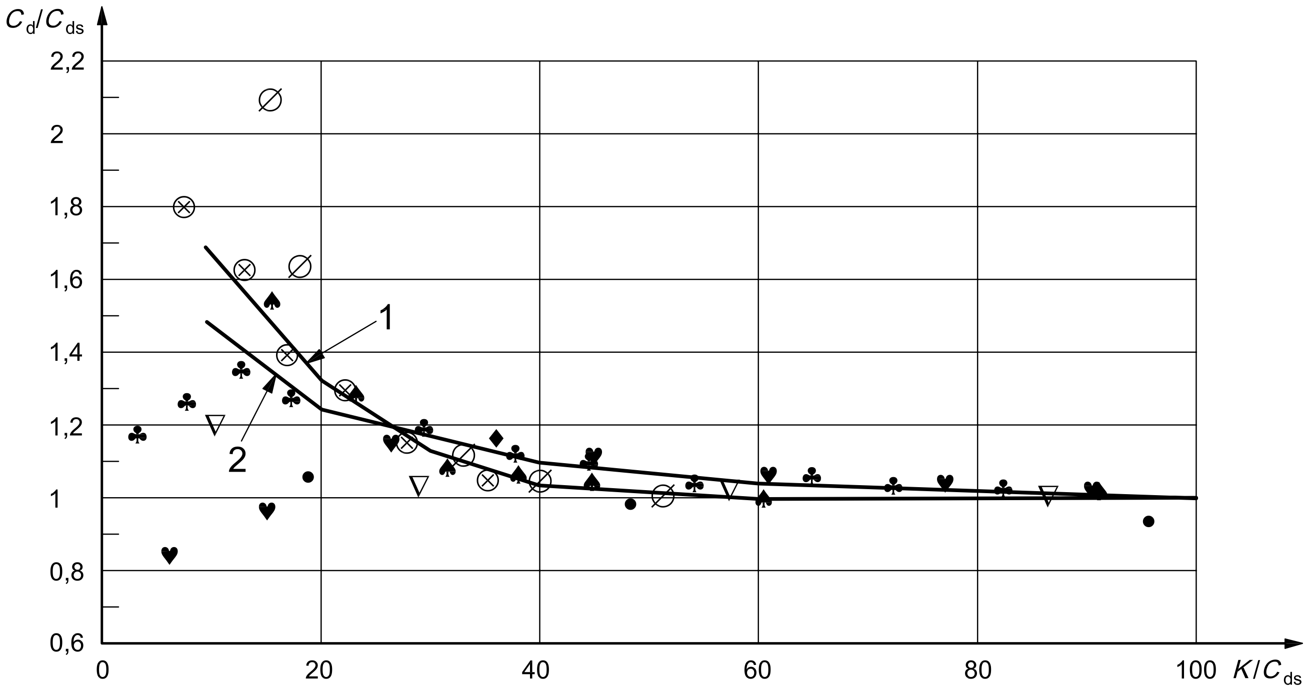
Figure A.9.5-4 shows drag coefficient data for $K < 12$, which are more appropriate for calculating hydrodynamic actions on nearly vertical members in less extreme sea states and drag damping in earthquake-excited motion. For $K < 12$, the smooth and rough cylinder data are similar if K is not normalized by C_{ds}. The data in Reference [A.9.5-18] do not agree well with the curves in Figure A.9.5-4, presumably because of the relatively low Reynolds numbers in the tests for the lowest values of K and because of the lack of wave spreading in the tests for the higher values of K.

The symbols shown in Figure A.9.5-3 do not represent individual data points. Rather, they represent values from a curve fitted through a scatter of data points. When designing a structure consisting of a single isolated column, the scatter in the C_d data should be considered. The data in Reference [A.9.5-18] for one-dimensional, sinusoidally oscillating motion, which are notably omitted from Figure A.9.5-3, represent a reasonable upper bound. For a structure consisting of many members, the scatter in C_d may be neglected, as the deviations from the mean curve are uncorrelated from member to member, see Reference [A.9.5-19].

Figures A.9.5-5 and A.9.5-6 show data for the inertia coefficient, C_m, for a nearly vertical circular cylinder. Figure A.9.5-5 shows that C_m for both smooth and rough cylinders approaches the theoretical value of 2,0 for $K \leqslant 3$. For $K > 3$ with the onset of flow separation, C_m begins to decrease. With the exception of rough cylinder data given in Reference [A.9.5-18], which exhibit a pronounced drop ("inertia crisis") in C_m at $K \approx 12$, it appears that a single sloping line is adequate for both smooth and rough cylinders up to $K \approx 12$ — beyond which smooth and rough cylinder data begin to diverge. In Figure A.9.5-6, the single line from Figure A.9.5-5 is seen to split into two lines because K is divided by $C_{ds} = 0,66$ for smooth cylinders and $C_{ds} = 1,1$ for rough cylinders. The value of C_m is taken as 1,6 for smooth cylinders and 1,2 for rough cylinders for $K/C_{ds} \geqslant 17$.

Although Figures A.9.5-3 to A.9.5-6 are based on circular cylinder data, the figures are also applicable to non-circular cylinders, provided the appropriate value of C_{ds} is used and C_m is multiplied by $C_{m0}/2$, where C_{m0} is the theoretical value of C_m for the non-circular cylinder as $K \to 0$.

Furthermore, while Figures A.9.5-3 to A.9.5-6 were developed for use with individual, deterministic waves, they can also be used for dynamic analysis (either in the time or in the frequency domain) of fixed structures by using significant wave height and spectral peak period to calculate K.



Key

K Keulegan-Carpenter number

C_d drag coefficient

C_{ds} steady-flow drag coefficient at post-critical Reynolds numbers

⊗ Heideman (1979), $C_{ds} = 1{,}00$

∅ Heideman (1979), $C_{ds} = 0{,}68$

♠ Bishop (1985), $C_{ds} = 0{,}66$

♣ Rodenbusch (1983), $C_{ds} = 1{,}10$ random directional

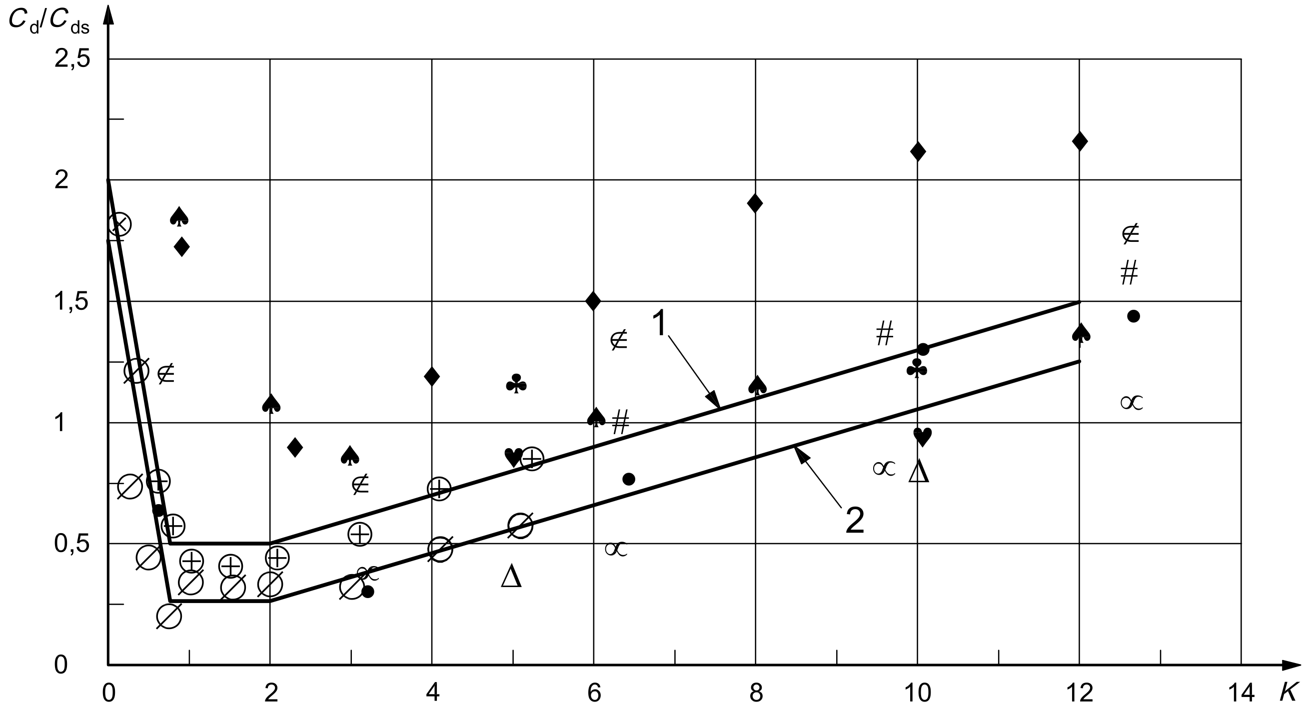
♥ Rodenbusch (1983), $C_{ds} = 0{,}66$ random directional

∇ Rodenbusch (1983), $C_{ds} = 1{,}10$ figure 8 orbits

• Rodenbusch (1983), $C_{ds} = 0{,}66$ figure 8 orbits

♦ Ohmart & Gratz (1979), $C_{ds} = 0{,}66$ figure 8 orbits

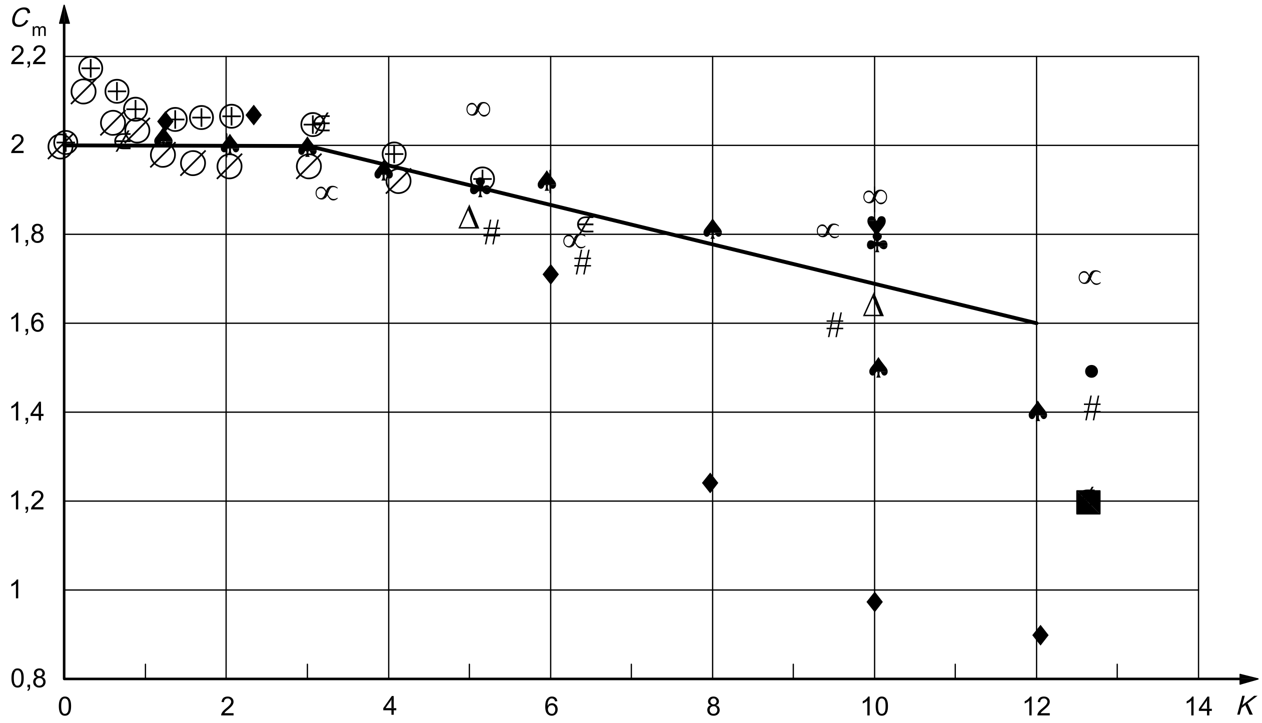
Figure A.9.5-3 — Wake amplification factor for drag coefficients as function of K/C_{ds}



Key

1 rough member (C_{ds} = 1,2)
2 smooth member (C_{ds} = 0,6)
K Keulegan-Carpenter number
C_d drag coefficient
C_{ds} steady-flow drag coefficient at post-critical Reynolds numbers
♣ Rodenbusch (1983), C_{ds} = 1,10 random directional
♥ Rodenbusch (1983), C_{ds} = 0,66 random directional
Δ Bearman (1985), C_{ds} = 0,60
∉ Rodenbusch (1983), C_{ds} = 1,10 sinusoidal
• Rodenbusch (1983), C_{ds} = 0,66 sinusoidal
♦ Sarpkaya (1986) C_{ds} = 1,10
♠ Sarpkaya (1986) C_{ds} = 0,65
∝ Garrison (1990) C_{ds} = 0,65
⊕ Marin (1987) C_{ds} = 1,10
Garrison (1990) C_{ds} = 1,10
∅ Marin (1987) C_{ds} = 0,60
⊗ Iwaki (1991) C_{ds} = 1,10

Figure A.9.5-4 — Wake amplification factor for drag coefficients as function of K



Key

K Keulegan-Carpenter number

C_m inertia coefficient

♣ Rodenbusch (1983), C_{ds} = 1,10 random directional

♥ Rodenbusch (1983), C_{ds} = 0,66 random directional

Δ Bearman (1985), C_{ds} = 0,60

∉ Rodenbusch (1983), C_{ds} = 1,10 sinusoidal

∞ Bishop (1985), C_{ds} = 0,66 random directional

• Rodenbusch (1983), C_{ds} = 0,66 sinusoidal

♦ Sarpkaya (1986) C_{ds} = 1,10

♠ Sarpkaya (1986) C_{ds} = 0,65

∝ Garrison (1990) C_{ds} = 0,65

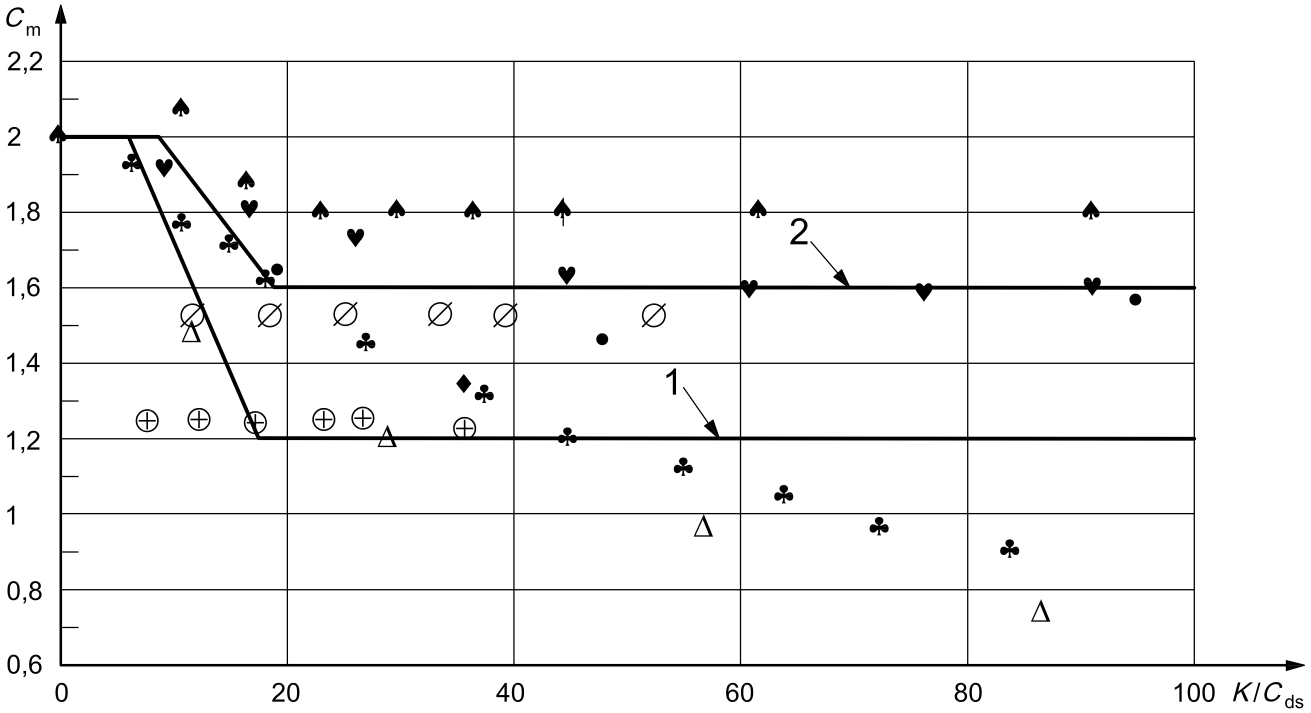
⊕ Marin (1987) C_{ds} = 1,10

Garrison (1990) C_{ds} = 1,10

∅ Marin (1987) C_{ds} = 0,60

⊗ Iwaki (1991) C_{ds} = 1,10

Figure A.9.5-5 — Inertia coefficient as function of K



Key

1 rough member (C_{ds} = 1,1)
2 smooth member (C_{ds} = 0,66)
K Keulegan-Carpenter number
C_{ds} steady-flow drag coefficient at post-critical Reynolds numbers
C_m inertia coefficient
♣ Rodenbusch (1983), C_{ds} = 1,10 random directional
♥ Rodenbusch (1983), C_{ds} = 0,66 random directional
Δ Rodenbusch (1983), C_{ds} = 1,10 sinusoidal
• Rodenbusch (1983), C_{ds} = 0,66
♦ Ohmart & Gratz (1979) C_{ds} = 0,60
♠ Bishop (1985) C_{ds} = 0,66
⊕ Heideman (1979) C_{ds} = 1,00
∅ Heideman (1979) C_{ds} = 0,68

Figure A.9.5-6 — Inertia coefficient as function of K/C_{ds}

A.9.5.2.3.5 Current/wave velocity ratio

The effect of a steady in-line current added to oscillatory motion is to push C_d toward C_{ds}, its steady flow value. Data show that, for practical purposes, $C_d = C_{ds}$ when the current/wave velocity ratio (r) is greater than 0,4. For $r << 0,4$, the effect of a steady in-line current can be accommodated by modifying the Keulegan-Carpenter number. A first order correction would be to multiply K due to wave alone by

$$1 + 1{,}6\,r + 0{,}8\,r^2 \qquad \text{for } 0 < r < 0{,}4 \qquad \text{(A.9.5-1)}$$

A current component normal to the wave direction also drives C_d toward C_{ds}, since it reduces the impact of wake encounter. Data show that, for practical purposes, $C_d = C_{ds}$ for $V_N\,T_i/C_{ds}\,D > 8$. On the other hand, wake encounter has nearly its full impact for $V_N T_i/C_{ds}\,D < 1$. Here, V_N is the component of the current velocity normal to the wave direction and T_i is the intrinsic wave period.

A.9.5.2.3.6 Member orientation

For members that are not nearly vertical, the effect of wake encounter, as characterized by the K dependence shown in Figures A.9.5-3 to A.9.5-6, is small. For horizontal and diagonal members, it is sufficient to use the theoretical value of C_m at $K \to 0$ and the steady-flow value of $C_d = C_{ds}$ at $K \to \infty$.

A.9.5.2.4 Current blockage

Taking account of current blockage can be of benefit to the design of space frame structures with a large number of conductors.

In general, no space frame or lattice type structure is totally transparent to waves and current. These structures cause a global distortion of the incident waves and of the current in and around the structure. Since global hydrodynamic action is calculated by summing hydrodynamic actions on individual members, it is important that the local incident flow be used to calculate hydrodynamic action on individual members in Morison's Equation (9.5-1) to account for global distortion effects.

Space frame structures distort the waves, as well as the current. Some field data indicate that the root-mean-square wave orbital velocity very near the structure is slightly reduced from that which is several structure widths up-wave. However, this reduction is not evident in all the data. Until more evidence to the contrary is accumulated, it is appropriate to continue with the assumption that a typical space frame structure does not significantly distort the incident wave kinematics in a global sense.

For currents, however, there now exists a substantial body of evidence that supports a reduction of the current within space frame structures relative to the free stream current. The amount of blockage depends on the type of structure. For dense structures it will be large, while for very transparent structures it will be small. Laboratory and field data indicate that the blockage factor can be as low as 0,6 for a dense structure (e.g. the Lena guyed tower[A.9.5-20]), approximately 0,7 for a typical compliant tower[A.9.5-21], and approximately 0,75 to 0,85 for a typical space frame structure[A.9.5-22]; see Table A.9.5-1. Figure A.9.5-7 shows the measured current field at 18 m depth around and through the Bullwinkle platform during a Loop Current event in the Gulf of Mexico in 1991[A.9.5-23]. The average blockage factor within the structure computed from the data is 0,77.

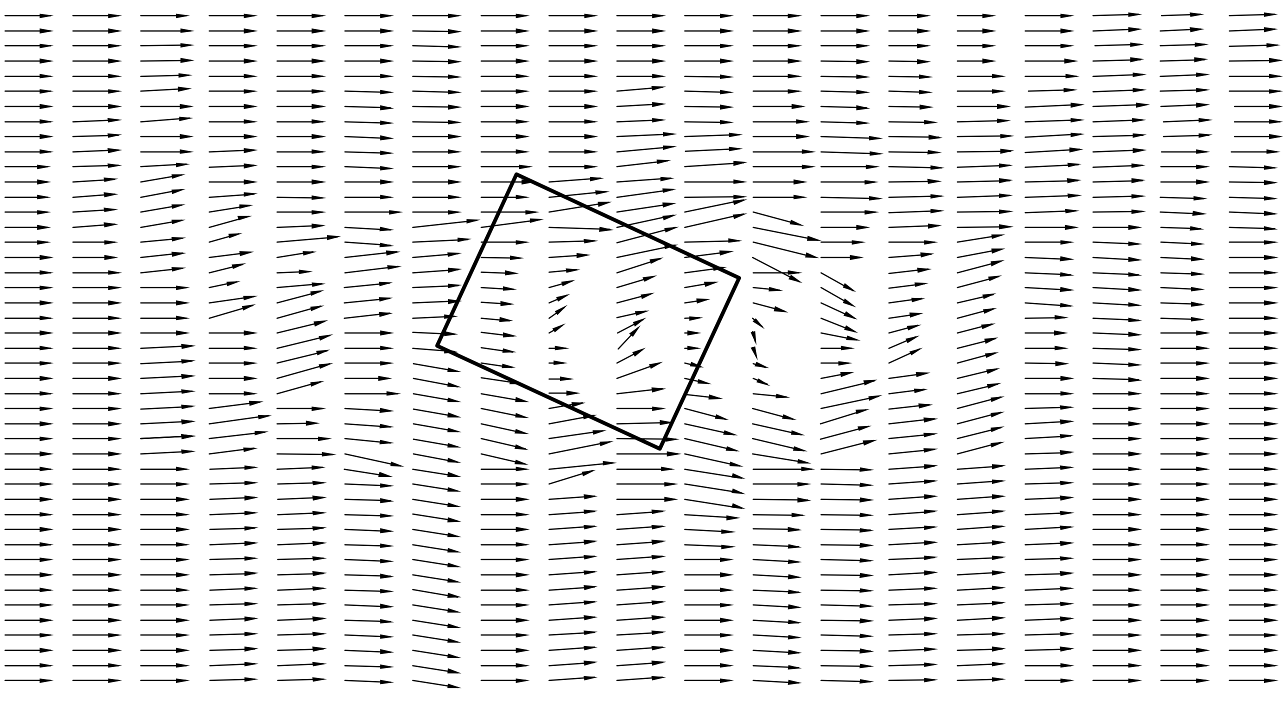


Figure A.9.5-7 — Current blockage flow — Current vectors computed from Doppler measurements

Table A.9.5-1 — Approximate factors for typical tubular space frame structures

Number of legs	Wave heading	Current blockage factor
3	all	0,90
4	end-on	0,80
	diagonal	0,85
	broadside	0,80
6	end-on	0,75
	diagonal	0,85
	broadside	0,80
8	end-on	0,70
	diagonal	0,85
	broadside	0,80

For structures with other configurations or structures with an atypical number of conductors, a current blockage factor can be calculated using the method described below. Factors calculated to be less than 0,7 should not be used without empirical evidence to support them. For free-standing or braced caissons and tripod structures, the current blockage factor should be 1,0.

The blockage factor for steady current can be estimated from the "actuator disk" model[A.9.5-23] as

$$\left[1+\sum_i (C_\mathrm{d}A)_\mathrm{i}/4\bar{A}\right]^{-1} \qquad \text{(A.9.5-2)}$$

where

$\sum_i (C_\mathrm{d}A)_\mathrm{i}$ is the summation of the "drag areas" of all members (including horizontal members) in the flow;

$\bar{A}$ is the area within the perimeter area of the structure projected normal to the current.

For structures where geometry changes significantly with depth, the blockage factor can be computed for different depth levels. If the calculated reduction factor is less than 0,7, consideration should be given to modelling the structure as a series of actuator disks rather than a single actuator disk.

An alternative expression for the blockage factor based on a similar approach, but accounting for mixing downstream, is given in Reference [A.9.5-24]. In the case of small values of the ratio $\sum_i (C_\mathrm{d}A)_\mathrm{i}/\bar{A}$, the alternative expression reduces to Equation (A.9.5-2).

The terms *global blockage*, discussed here, and *shielding*, discussed in 9.5.2.5, are related and are sometimes used to describe the same phenomenon, as is the term *interference*. In this International Standard *shielding* is used only with reference to members in the local wake of neighbouring members (such as conductor arrays), and the shielding factor should be applied to the calculated hydrodynamic action due to both waves and currents. *Blockage* is used in relation to the entire structure, and the blockage factor should be applied to the far-field current velocity only. With this distinction, the blockage factor is first used to calculate a reduced current velocity for the entire structure. The reduced current velocity and undisturbed wave kinematics are then used in Morison's equation to calculate local hydrodynamic actions on all members. The calculated hydrodynamic action on conductors would then be further reduced by the shielding factor.

A.9.5.2.5 Conductor shielding factor

The empirical basis for the shielding factor for the hydrodynamic action on conductor arrays is shown in Figure A.9.5-8. Data from flow directions perfectly aligned with a row or column of the array are excluded for conservatism.

The data in Figure A.9.5-8 are from steady flow tests and oscillatory flow tests at very high amplitudes of oscillation. Thus, the factor is strictly applicable only in a steady current with negligible waves or near the mean water level in very large waves. The data in Reference [A.9.5-25] indicate that the factor is applicable if $A/S > 6$, where A is the amplitude of oscillation and S is the centre-to-centre spacing of the conductors in the wave direction.

The data in Reference [A.9.5-26] indicate that the range of applicability can be expanded to $A/S > 2{,}5$. For lower values of A/S, there is still some shielding until $A/S < 0{,}5$ [A.9.5-25]. With $A \approx U_{m0}\, T_i/2\pi$, where U_{m0} is defined in A.9.5.2.3 and T_i is the intrinsic wave period, respectively, the approximate shielding regimes are as follows:

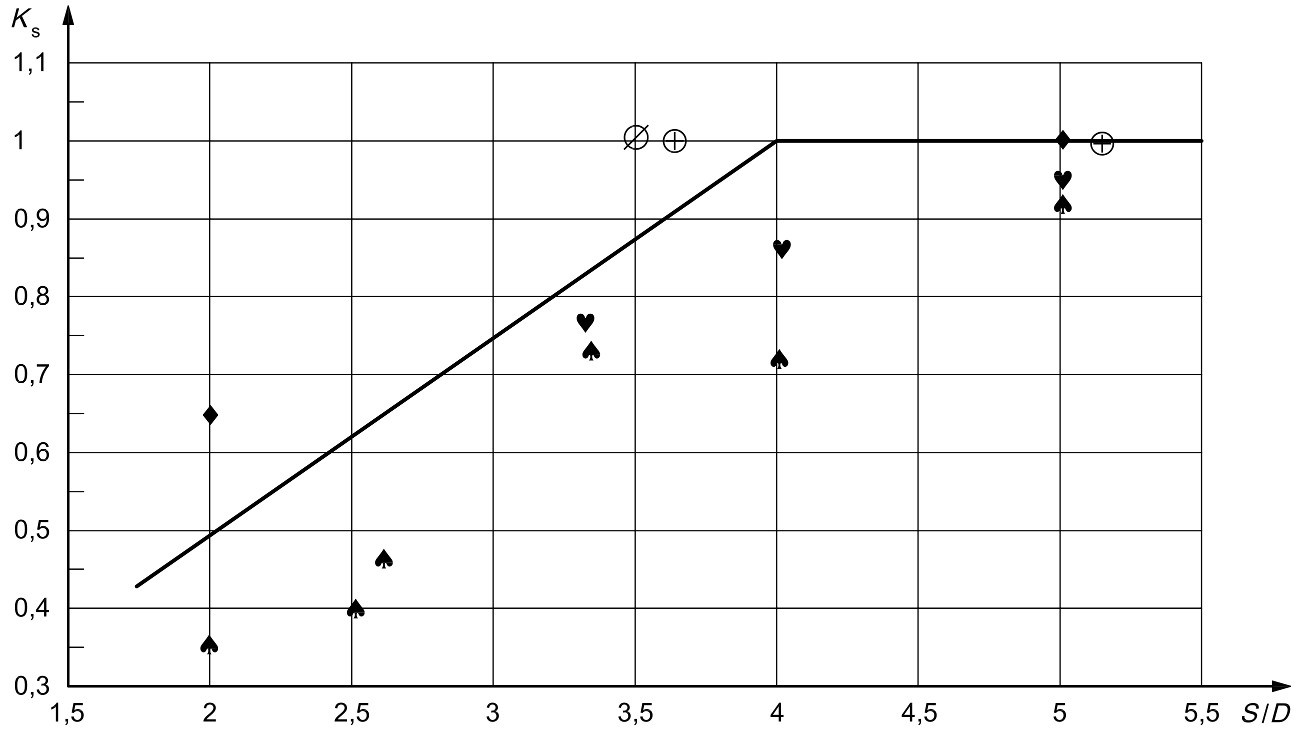
a) $A/S > 2{,}5$, asymptotic shielding, factor from Figure A.9.5-8;

b) $0{,}5 < A/S < 2{,}5$, partial shielding;

c) $A/S < 0{,}5$, no shielding, factor $= 1{,}0$.

In the absence of better information, the shielding factor in the partial shielding regime for any value of S/D may be linearly interpolated as a function of A/S, between the factor from Figure A.9.5-8 for $A/S = 2{,}5$ and 1,0 for $A/S = 0{,}5$. Waves considered in fatigue analyses can lie in the partial shielding regime.

NOTE See also A.9.5.2.4 for the relationship of shielding to current blockage.



Key

S conductor spacing
D conductor diameter
K_s shielding factor
♥ Reed (1990) waves (K = 126)
♦ Heideman (1985) waves (K = 250) and current
♠ Reed (1990) current
⊕ Beckman (1979) waves and current
∅ Sterndorff (1990) waves

Figure A.9.5-8 — Shielding factor for actions due to waves on conductor arrays as function of conductor spacing

A.9.5.3 Hydrodynamic models for appurtenances

The hydrodynamic model of a structure is used for the calculation of hydrodynamic actions that represent the global hydrodynamic action on the actual structure. The model does not explicitly need to include every component of the structure, provided the dimensions and/or hydrodynamic coefficients of the components included account for the contribution of the omitted components to the global hydrodynamic action. The hydrodynamic model should account for the effects of marine growth and for flow interference effects (blockage and shielding) where appropriate.

Appurtenances include components such as boat landings, fenders or bumpers, walkways, stairways, grout lines and anodes. Although it is beyond the scope of this annex to provide modelling guidance for every appurtenance, some general guidance is provided.

Boat landings generally consist of a large number of closely spaced tubular members. If the components are modelled individually, shielding effects, depending upon the wave direction, may be considered in a similar manner to those for conductor arrays. Alternatively, boat landings may be modelled as either rectangular solids or as one or more plates, with directionally dependent hydrodynamic properties. Guidance for hydrodynamic coefficients for solid shapes and plates can be found in Reference [A.9.5-15].

Conductor guide frames may also be modelled as rectangular solids or, if appropriate, as plates. In both cases, different coefficients are appropriate for vertical and horizontal fluid flow.

Large fenders or boat bumpers and their supports are usually modelled as individual members. They may be treated as non-structural members, provided that their design has been shown to be adequate for their intended purpose. Walkways, stairways and grout lines may be modelled using equivalent circular members, although they may be ignored where experience has shown this to be acceptable.

The treatment of anodes depends upon their number and size. Anodes may be modelled as equivalent circular cylinders. Alternatively, hydrodynamic actions on anodes may be approximated by increasing the diameters and/or hydrodynamic coefficients of the components to which they are attached.

A.9.6 Actions caused by current

No guidance is offered.

A.9.7 Actions caused by wind

A.9.7.1 General

No guidance is offered.

A.9.7.2 Determining actions caused by wind

Shielding and shape coefficients for individual components, modules and lattice structures are described in Reference [A.9.7-1].

A.9.7.3 Wind actions determined from models

No guidance is offered.

A.9.8 Equivalent quasi-static action representing dynamic response caused by extreme wave conditions

A.9.8.1 General

The conditions specified in 9.8 are generally not satisfied in comparatively light sea states. However, while dynamic response in light sea states can be important for checking of fatigue, it will generally not govern structural design for strength. Therefore, the principle of using an equivalent quasi-static action may, under the conditions stated, be applied to extreme environmental conditions.

A.9.8.2 Equivalent quasi-static action (D_e) representing the dynamic response

The equivalent quasi-static action, D_e, is determined at a global level and subsequently distributed over the height of the structure to produce the approximately correct extreme dynamic base shear and overturning moment in the design sea state. Structural design may then proceed as for a quasi-statically responding structure, using design actions that are modified to account for dynamics. This makes design of dynamically responding structures in accordance with the conditions specified in 9.8.1 practical and yet entirely adequate.

The magnitude of D_e should be such that the probability of the total extreme dynamic response due to ($E_{wce} + D_e$) is approximately the same as that of the quasi-static response due to E_{wce} alone. The conditions given in 9.8.1 ensure that phase differences between the dynamic and the quasi-static responses are small enough to assume that this is achieved.

For structures with a sufficiently short fundamental natural period, when compared to the predominant excitation periods present in extreme wave conditions, D_e may be neglected entirely. However, dynamic response depends on several factors and it is difficult to make a universal statement as to when dynamic response is negligibly small. Generally speaking, dynamic response increases as the fundamental natural period of the structure increases and moves closer to the period of the peak of the design wave spectrum, thereby entering a range of periods where the wave spectrum has noticeable energy. Studies, as well as a history of successful performance, indicate that fixed structures of space frame configuration with fundamental natural periods shorter than 2,5 s to 3 s will experience only a few percent dynamic amplification in the design sea state. For such structures, the dynamic amplification may be neglected. In accordance with the requirements of 9.8.1, this rule of thumb hence assumes peak spectral periods for design sea states of 13 s or longer. If the peak period of the design wave spectrum for such structures is shorter, the possible influence of dynamics should be evaluated. However, these observations should not be generalized to other types of structure or to other design situations.

NOTE 1 See 12.5 for further information and guidance on dynamically responding structures and how dynamic behaviour should be treated.

The dynamic response can be influenced by the following, in addition to its dependence on the ratio of the fundamental natural period of the structure to the period of the peak of the design wave spectrum.

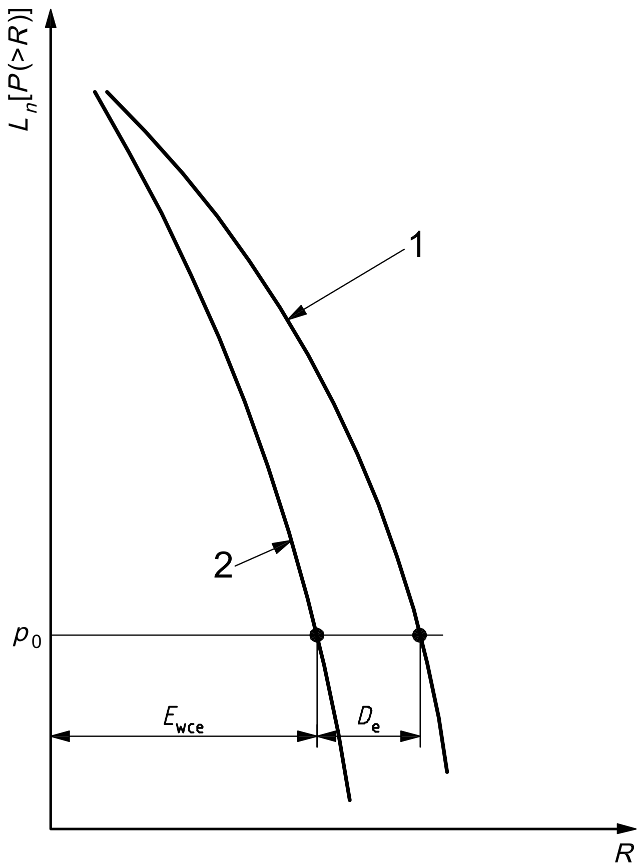
a) Cancellation or reinforcement of wave action due to the spatial separation of members in the space frame, particularly between the legs. Structures that do not benefit from this effect, such as small structures with closely spaced legs or one leg only, are therefore potentially more susceptible to dynamic response from short period waves than large space frame structures. For these cases, the conditions given in 9.8.1 will generally not be satisfied and the influence of dynamics will again have to be assessed more carefully in accordance with Clause 12.

   NOTE 2 When directional spreading is taken into account, cancellation/reinforcement effects are reduced.

b) Higher harmonic frequency components of the hydrodynamic action originating from non-linear drag actions and free surface inundation effects. These effects require special evaluation if significant.

The indirect approach presented in 9.8.2 b) for the determination of D_{e} uses a global dynamic analysis to estimate the dynamic amplification factor, $k_{\mathrm{DAF,BS}}$, for base shear, where $k_{\mathrm{DAF,BS}}$ is the ratio of the dynamic base shear to the static base shear. In time domain analyses, $k_{\mathrm{DAF,BS}}$ can be estimated by taking the average of the values of $k_{\mathrm{DAF,BS}}$ for a few dozen short wave segments that include the design wave. A robust method of deducing $k_{\mathrm{DAF,BS}}$ from time domain analyses is to plot order statistics of dynamic response and quasi-static response to the same realization of the design sea state. The ratio of the dynamic to quasi-static response, at the level of the exceedance probability of the design wave in the sea state, can be used to obtain a reliable estimate of $k_{\mathrm{DAF,BS}}$.

Alternatively, the calculation may be taken into long-term statistics; the curves in Figure A.9.8-1 may be estimated by simulation and then $k_{\mathrm{DAF,BS}}$ taken off directly as $(E_{\mathrm{wce}} + D_{\mathrm{e}})/E_{\mathrm{wce}}$. For truly linear problems and frequency domain methods, $k_{\mathrm{DAF,BS}}$ reduces to the ratio of the root-mean-square (rms) dynamic to the rms static base shear response when there is no current.



Key

1 with dynamics

2 no dynamics

R global response

Figure A.9.8-1 — Definition of D_e

D_e should next be distributed over the height of the structure to form a set of inertial actions, d_k, that is added to the environmental actions at the elevations, k. This distribution may be done in proportion to the fundamental mode shape of the structure. The procedure is as follows:

$$D_e = \sum_{k=1}^{K} d_k = \sum_{k=1}^{K} \alpha_1 \omega_1^2 m_k \phi_{1k} = \alpha_1 \omega_1^2 \sum_{k=1}^{K} m_k \phi_{1k} \quad \text{(A.9.8-1)}$$

$$d_k = \alpha_1 \omega_1^2 m_k \phi_{1k} = \frac{m_k \phi_{1k}}{\sum_{k=1}^{K} m_k \phi_{1k}} \cdot D_e \quad \text{(A.9.8-2)}$$

where

D_e is the extreme dynamic base shear minus the extreme quasi-static base shear;

d_k is the inertial action at elevation k ($k = 1, 2, \dots K$);

α_1 is a scale factor for the first mode shape (having the dimension of a length);

ω_1 is the first natural frequency of the structure;

m_k is the lumped mass at elevation k;

ϕ_{1k} is the non-dimensional displacement in first mode shape at elevation k.

This results in a set of distributed actions, d_k, that reproduces the dynamic base shear, but generally not the dynamic overturning moment at the same time.

The above procedure for the equivalent quasi-static base shear may also be applied to the equivalent quasi-static overturning moment M_e to determine an analogous distribution. This results in Equations (A.9.8-3) and (A.9.8-4):

$$M_\mathrm{e} = \sum_{k=1}^{K} d_k\ h_k = \sum_{k=1}^{K} \beta_1\ \omega_1^2\ m_k\ \phi_{1k}\ h_k = \beta_1\ \omega_1^2 \sum_{k=1}^{K} m_k\ \phi_{1k}\ h_k \qquad \text{(A.9.8-3)}$$

$$d_k = \beta_1\ \omega_1^2\ m_k\ \phi_{1k} = \frac{m_k\ \phi_{1k}}{\sum_{k=1}^{K} m_k\ \phi_{1k}\ h_k} \cdot M_\mathrm{e} \qquad \text{(A.9.8-4)}$$

where, additionally,

M_e is the extreme dynamic overturning moment minus the extreme quasi-static overturning moment;

h_k is the height of mass point, m_k, above the sea floor;

β_1 is a scale factor for the first mode shape (having the dimension of a length).

As the scale factors, α_1 and β_1, are usually not the same, the dynamic overturning moment is now correctly reproduced, but the dynamic base shear is not.

Using both the magnitudes of D_e and M_e, a linear combination of the first and the second mode may be used to determine a distribution of inertial actions such that both the dynamic base shear and the dynamic overturning moment are reproduced by the set of inertial actions. This is done in the following manner:

$$D_\mathrm{e} = \sum_{k=1}^{K} d_k = \sum_{k=1}^{K} \left(\gamma\ \omega_1^2\ m_k\ \phi_{1k} + \delta\ \omega_2^2\ m_k\ \phi_{2k} \right) = \gamma\ \omega_1^2 \sum_{k=1}^{K} \left(m_k\ \phi_{1k} \right) + \delta\ \omega_2^2 \sum_{k=1}^{K} \left(m_k\ \phi_{2k} \right) \qquad \text{(A.9.8-5)}$$

$$M_\mathrm{e} = \sum_{k=1}^{K} d_k\ h_k = \sum_{k=1}^{K} \left(\gamma\ \omega_1^2\ m_k\ \phi_{1k}\ h_k + \delta\ \omega_2^2\ m_k\ \phi_{2k}\ h_k \right) = \gamma\ \omega_1^2 \sum_{k=1}^{K} \left(m_k\ \phi_{1k}\ h_k \right) + \delta\ \omega_2^2 \sum_{k=1}^{K} \left(m_k\ \phi_{2k}\ h_k \right) \qquad \text{(A.9.8-6)}$$

$$d_k = \gamma\ \omega_1^2\ m_k\ \phi_{1k} + \delta\ \omega_2^2\ m_k\ \phi_{2k} \qquad \text{(A.9.8-7)}$$

where, additionally,

γ, δ are scale factors for the first and the second mode shape, respectively (having the dimension of a length);

ω_2 is the second natural frequency of the structure;

ϕ_{2k} is the non-dimensional displacement in second mode shape at level k.

The two unknown scalars, γ and δ, are determined by solving the two Equations (A.9.8-5) and (A.9.8-6) for D_e and M_e simultaneously, after which the set of inertial actions, d_k, can be determined from Equation (A.9.8-7).

A.9.8.3 Global dynamic analysis in waves

A.9.8.3.1 General

The dynamic analysis is performed at a global level, for which purpose a suitably simplified structural model can be used as appropriate. Subclause A.9.8.3 gives some guidance for performing the dynamic analysis.

A.9.8.3.2 Dynamic analysis methods

No guidance is offered.

A.9.8.3.3 Design sea state

Random waves are generally modelled as a linear superposition of a large number of sinusoidal (Airy) components, each with different amplitude, frequency and phase. For dynamic analyses, random waves can be modelled either in the time domain or in the frequency domain. Time domain simulation may be considered the most general of the methods because non-linearities can be directly included. Results from such simulations therefore serve as a reference with which other approaches can be compared. Non-linearities in the environmental actions arise from the squared drag term in Morison's equation and the inundation effects as the wave surface moves from trough to crest. Non-linearities from the structural system come from a non-linear foundation, and from large displacements and yielding of members and joints. When linearization of these non-linearities can be justified, frequency domain analysis provides a very efficient and easy-to-interpret means of using random waves for the determination of global k_{DAF} factors.

In the frequency domain, each frequency component is considered separately, one at a time, and the response of the structure to that frequency component is determined. In this manner, a "frequency response function" is established, which is then multiplied by the wave spectrum of the design sea state in order to obtain the response spectrum. For linear problems, this frequency response function is independent of the amplitude chosen for each wave component. The influence of non-linearities may be included in an approximate manner if each wave component is modelled with a realistic finite steepness. However, for a finite steepness, the linear (Airy) wave kinematics theory that applies below the mean still water level should be modified accordingly. Extrapolation of the theory to elevations above the still water level results in particle velocities that are (much) too high. Adequate models that are commonly employed use vertical extrapolation above water level, Wheeler (linear) stretching[A.9.8-1], and delta stretching[A.9.8-2]; see also ISO 19901-1.

NOTE *Frequency response function* is the most correct name for the response of a linear system to harmonic excitation: it is a complex-valued function of frequency having magnitude (the modulus) and phase (the argument). The frequency response function is often also called *transfer function* or *response amplitude operator* (RAO). In general, the transfer function also has magnitude and phase, but in many cases it refers more particularly to the magnitude (the modulus) only. RAO is by definition a scalar function associated with the modulus only.

In the time domain, the response of the structure to the simultaneously applied frequency components is determined in the form of a time series. Each component should be given an appropriate amplitude and phase before they are combined. Typically, the amplitudes are randomly selected from a Rayleigh distribution in accordance with the discretized design wave spectrum, while phases may be randomly selected from a uniform distribution over 0° to 360°. A wave model thus derived is referred to as a simulation model. An alternative method is to determine the amplitude and phase of each component from a Fourier analysis of a measured wave elevation trace that corresponds with the design wave spectrum, if such a measured wave trace is available. Such a wave model is referred to as a *conditional* model. Other methods for selecting amplitudes and phases are the subject of research. Both a simulation and a conditional model require modification of the linear (Airy) kinematics, as with the frequency domain approach. A weakness of the simulation model as it is usually applied is that it produces a Gaussian water surface elevation and does not reproduce the skewness observed in real wave records (higher, sharper crests and shallower, flatter troughs). The conditional model incorporates these effects to some extent but cannot faithfully reproduce simultaneous wave traces at spatially separated points either, because the linear dispersion relationship does not propagate the components correctly. Random wave theories that account for the non-linear interactions between frequency components and the consequent effects on wave kinematics and wave propagation are under development[A.9.8-3]. Methods that include directional spreading of wave energy are now available.

It should further be recognized that a time domain simulation is a single realization (a sample of finite length) of the full population of an infinite number of possible realizations of infinite length that represent the underlying random process, regardless of whether a simulation or a conditional wave model is used. The interpretation of the results of one (or a few) time domain simulations (samples) and their use to derive statistical properties that provide correct and unbiased results for the underlying random process is far from straightforward. If these difficulties are not carefully managed, the potential error associated with the use of time domain simulations may well be larger than the error involved in linearization to enable frequency domain methods to be applied. When time domain simulation is applied, the use of order statistics as mentioned in A.9.8.2 is strongly recommended.

A.9.8.3.4 Hydrodynamic action on a member

For fixed structures, the relative velocity formulation of Morison's equation should not be used for regular or random waves. When the displacement of the structure is less than the diameter of a representative member and the motion of the structure is at a high frequency compared to the frequency(ies) of the wave(s), the motion of an individual member interacts with its own wake and does not develop the hydrodynamic actions predicted by the relative velocity formulation of Morison's equation (see A.9.5.2.3 and, for example, Reference [A.9.8-4]). Using relative velocity would have the same effect on the dynamic analysis as using too large a value of the damping. In this situation, as the damping force and the environmental action do not interact, the use of a formulation with absolute water particle velocities is more realistic. The associated viscous damping coefficient should be estimated in the absence of waves.

A.9.8.3.5 Mass

No guidance is offered.

A.9.8.3.6 Damping

Estimating damping coefficients is difficult; reliable data for relevant circumstances are extremely scarce if available at all. However, for the conditions to which 9.8 applies, dynamic response is stiffness controlled, thus the influence of damping is moderated and the selection of a damping coefficient is not critical to the results. The value of 2 % to 3 % can be slightly conservative for extreme wave conditions but has little influence on D_e. For the lighter sea states that are relevant for fatigue analyses, the damping coefficient should be reduced to a value of 1 % to 2 % of critical.

A.9.8.3.7 Stiffness

For fixed structures a linear stiffness model is normally sufficient; second order geometrical stiffness effects (so-called P-Δ effects) are only relevant for more flexible structures and do not normally need to be included in the global dynamic analysis. Linearization of the foundation stiffness, if required, should be performed at a suitable response level reflecting the response in the extreme sea state. For fatigue analyses it is generally appropriate to consider a lower response level and a correspondingly stiffer foundation.

When the objective is to determine a set of inertial actions for use with a non-linear ultimate strength analysis, non-linear stiffness properties should be maintained.

A.9.9 Factored actions

A.9.9.1 General

No guidance is offered.

A.9.9.2 Factored permanent and variable actions

Partial action factors for permanent and variable actions were determined through a calibration process that also included wind, wave and current actions. No distinction was made between G_1 and G_2 or Q_1 and Q_2, so the partial action factors are the same. Nevertheless, it is useful to establish these subsets, in which the actions have common degrees of uncertainty and duration of occurrence.

The calibration process resulted in a choice of 1,3 for the partial action factor on permanent actions and of 1,5 for the partial action factor on variable actions. These choices result in a larger component safety index when using load and resistance factor design (LRFD) practice than when using working stress design (WSD) practice for components with both environmental and permanent actions. While the reliability concept of uniform component safety could justify lower factors, the results would be a wide departure from past practice.

The component safety index calibration used the following statistical model: Q_1 and Q_2 have a coefficient of variation (COV) of 14 % and the mean value is the nominal value. G_1 and G_2 have a COV of 8 % and the mean value is the nominal value. For the calibration, the internal force due to permanent and variable actions was assumed to be 25 % of $(G_1 + G_2)$ and 75 % of $(Q_1 + Q_2)$.

A.9.9.3 Factored extreme environmental action

A.9.9.3.1 General

In the API LRFD[A.9.1-1], the same partial action factor was to be applied to environmental wind and wave effects. However the calibration of the partial action factor considered only the wave parameters. In the NBS-ANSI study[A.9.9-1], [A.9.9-2], the bias and COV for maximum lifetime wind effects, was found to be 0,78 and 37 % —very similar to the 0,7 and 37 % used for the wave effect parameters. This fact plus the correlation of high winds with high waves justifies the same partial action factor for wind and waves.

Where no information on partial action factors that are specific to the case under consideration is available, these factors may be taken to be $\gamma_{f,E} = 1{,}35$ and $\gamma_{f,D} = 1{,}25$ in accordance with the factors that were proposed in Reference [A.9.9-3], based on calibration work done for the Gulf of Mexico. In subsequent calibration work along similar lines, these factors were generally also found to be appropriate for some other areas. However, note the reservation expressed in the discussion in A.9.9.3.2.

A.9.9.3.2 General discussion of partial factors

The numerical values of the partial action factors (for example $\gamma_{f,E} = 1{,}35$ and $\gamma_{f,D} = 1{,}25$) as well as the resistance factors given in this International Standard have been largely derived on the basis of the original calibration work by Moses[A.9.9-3] to [A.9.9-7]. A calibration like this is suitable for an offshore province, such as the Gulf of Mexico, where the experience base is very extensive and satisfactory. At the same time, there is evidence that these structures are not overdesigned. For other geographical areas where such ample experience and evidence does not exist, calibration to the previous API WSD practice of that area is possible, but it does not achieve harmonization of safety levels worldwide, which is a major driver behind the initiative to develop this International Standard. For the specific case of actions due to extreme storms, it is known that the long-term distribution of environmental actions is a function of the geographic location. For structures with the same geometrical and structural properties, harmonization in safety levels hence requires location dependent partial action factors.

Reliability techniques can play a significant role in achieving harmonization, but such techniques need to be used differently from the calibration work referred to above in order to overcome the weaknesses of calibrations relative to the local WSD practice. This discussion lists the properties of more refined reliability models that may be used to address the above issues and to provide appropriate partial action factors for use with joint environmental conditions in different geographical locations. It is necessary that such models achieve results that are close to actuarial results rather than being notional results only.

Appropriate reliability models should include all of the following distinguishing features.

a) Description of the environment: the models take into account the joint probability of waves, currents and winds in defining the 100 year extreme as well as the complete long-term distribution of the extreme environmental action. For those aspects, both modelling and physical uncertainty will depend on the geographical area.

b) Wave load models: the models for actions due to waves incorporate a wave load recipe that accounts for the three-dimensional features of real seas and use realistic hydrodynamic coefficients.

c) Ultimate strength of the structural system rather than of individual components.

A.9.9.3.3 Partial action factor, $\gamma_{f,E}$

Reliability models and methods should be tested prior to their application in calculating new partial action factors or reserve strength ratios (RSRs). Their predictions of failure rates should be compared with historical values for a geographical area where there is experience of structural survival (and failure) in extreme storms and well-proven metocean data. Calculated values of the partial action factor and RSR should be compared with established values for that area. As well as testing the method, the exercise aids the engineering judgment of subsequent results.

The authors of Reference [A.9.9-8] concluded that reliability models with the features listed in A.9.9.3.2 could predict the failure rate of post-1977 structures in the Gulf of Mexico (GoM) with reasonable accuracy. If inaccurate, the models erred towards overprediction of failure rate. Such a reliability model has been used to estimate values of RSR and $\gamma_{f,E}$ that achieve a target level of safety in various environments[A.9.9-9]. A target probability of failure of 3×10^{-5} per year was considered to be appropriate for a new, permanently manned installation (because this level is small relative to the overall risk to personnel). The results obtained for the UK sector of the North Sea (NS) and the north-west shelf of Australia (AUS) are given in Table A.9.9-1.

Table A.9.9-1 — Values of partial action factor $\gamma_{f,E}$ and RSR to achieve target failure rate $P_f < 3 \times 10^{-5}$/yr for new manned installations (exposure level L1)

Environment	Partial action factor, $\gamma_{f,E}$	Mean RSR
AUS	1,59	2,18
NS	1,40	1,92

Partial action factors for north-west European waters have also been studied in References [A.9.9-10] and [A.9.9-11]. The relationship between partial action factor and RSR is reviewed in Reference [A.9.9-10]; the results suggest that smaller action factors are sufficient to achieve the RSRs given in Table A.9.9-1 and the target reliability.

For structures that are unmanned or evacuated in severe storms and where other consequences of failure are not very significant, a different safety level is appropriate. On the basis of Gulf of Mexico experience and generic cost benefit analysis, a probability of failure of less than 5×10^{-4} per year was chosen and used to generate the results in Table A.9.9-2. It is noted that a structure designed using these values is expected to be marginally more reliable than one designed according to Reference [A.9.9-12] in the Gulf of Mexico.

Table A.9.9-2 — Values of partial action factor, $\gamma_{f,E}$ and RSR to achieve target failure rate $P_f < 5 \times 10^{-4}$/yr for new unmanned installations (exposure level L2)

Environment	Partial action factor, $\gamma_{f,E}$	Mean RSR
AUS	1,17	1,60
GoM	1,17	1,60
NS	1,09	1,49

It is emphasized that the results in Table A.9.9-2 relate to new, unmanned (or evacuated) structures. For existing structures, the criteria may be relaxed, provided the risk is kept as low as reasonably practicable. API Task Group 92-5 has examined the category of existing structures at length as described in Reference [A.9.9-13] and guidelines for assessment of existing structures are given in Clause 24.

Some points should be borne in mind in relation to modelling uncertainty. Different areas can require different treatment of the modelling uncertainty to be associated with metocean data, both in relation to confidence in the design environmental action and the statistics of extreme actions. When greater modelling uncertainty is assumed (in any part of the calculation), it will tend to increase the partial factors required to achieve a given target reliability.

A.9.9.3.4 Partial action factor $\gamma_{f,D}$

D_e and E_e are assumed to act in the same direction, implying that the dynamic response is in phase with the applied environmental action. This is applicable to stiffness controlled dynamic response where the natural period is shorter than the predominant period of the applied loading, as discussed previously.

The 1,25 factor is based on a reliability analysis by Moses[A.9.9-14]. The calibration for the permanent actions and the extreme quasi-static environmental action was repeated with the addition of another random variable, the inertial partial action factor ($\gamma_{f,i}$). The random resistance was compared to the random internal force (S); thus S is due to the following F_d:

$$F_d = G + Q + E_e\,(1 + \gamma_{f,i}) \qquad \text{(A.9.9-1)}$$

where G, Q, E_e and $\gamma_{f,i}$ are all random variables.

Note that $(1 + \gamma_{f,i})$ is the ratio of the total dynamic (static and inertial) member internal force to the static member internal force due to only E_e. The variable $\gamma_{f,i}$ was taken to have a *COV* of 60 %. The mean value of $\gamma_{f,i}$ was taken to have a range of zero up to 0,8 to represent the fact that $\gamma_{f,i}$ values of individual members can be considerably larger than for a global response, such as base shear. The factor of 1,25 ensures that in any of these cases, the safety index (β) for a member is at least as great as the case when D_e equals zero.

The result of the calibration and the value of $\gamma_{f,D}$ obtained by it depends on the degree of detail and refinement of the dynamic analysis performed. Some experience indicates that $\gamma_{f,D}$ may be taken as 1,0, provided that the requirements of 9.8 and recommendations of A.9.8 are carefully followed.

A.9.10 Design situations

A.9.10.1 General considerations on the ultimate limit state

No guidance is offered.

A.9.10.2 Demonstrating sufficient RSR under environmental actions

No guidance is offered.

A.9.10.3 Partial factor design format

A.9.10.3.1 General

No guidance is offered.

A.9.10.3.2 Design actions for in-place situations

A.9.10.3.2.1 Design actions for operating situations

Where permanent and/or variable actions on the structure during temporary operating conditions are significantly more severe than normal maximum conditions (e.g. during hydro-test), it can be appropriate to define limiting environmental conditions in which the particular mode of operation may proceed. These limiting environmental conditions should then be applied when determining the environmental action for that mode of operation.

Typically, a 1 year to 5 year winter storm is used as an operating wind, wave and current condition in the Gulf of Mexico. In the South China Sea, for example, 1 year return period conditions have traditionally been used. Since these are mild by comparison with the extreme conditions, they do not have an unduly severe impact on the design. In the North Sea, however, 1 year environmental conditions are relatively severe by comparison with 100 year conditions, and their use would have an unduly large impact on design. In this area, the operating condition is most commonly considered in conjunction with specific activities undertaken on the platform, e.g. running drill pipe or crane operations. The operating environment is consequently determined as being the maximum condition in which the associated activities can be undertaken, see ISO 19901-3[2].

“Operating environmental conditions” combined with permanent and variable actions (and without a one-third allowable stress increase) has been a load case in API RP2A-WSD[A.9.10-1] since the first edition. This recognizes that some environmental action due to wave, wind and current will be present in combination with the maximum permanent and variable actions. For this reason, and because it is part of current Gulf of Mexico practice, an operating strength check was maintained in API RP2A-LRFD[A.9.10-2].

Unlike the environmental action due to extreme wind, wave and current conditions, the environmental action due to operating wind, wave and current conditions was intentionally not tied to a specific return period event in API RP2A-LRFD[A.9.10-2]. The partial factor calibration was performed without making use of the operating wind-wave-current case. The use of Equation (9.10-1) with E_o corresponding to “moderately severe conditions” in the Gulf of Mexico adds a small but uncalibrated increment to member size and safety for members loaded principally by permanent actions. Outside the Gulf of Mexico, if “moderately severe conditions” are defined as the same return period event, Equation (9.10-1) can be conservative beyond the original intention. Equation (9.10-1) can produce smaller design internal forces compared to API RP2A-WSD for certain cases, particularly for members that are subjected to mostly (but not 100 %) permanent actions in combination with an environmental action due to operating environmental conditions (E_o) that exceed approximately 0,5E_e.

In API RP2A LRFD, an overall partial action factor of 1,2 was selected to cover the analysis uncertainty that is associated with all action factors. Since the operational wind-wave-current check does not check safety for unusual environmental actions but rather primarily for permanent actions, a factor less than 1,35 is appropriate. To generalize the intent and this result in an approximate manner to geographical areas other than the Gulf of Mexico, an additional reduction factor on the environmental actions is introduced, giving $\gamma_{f,Eo} = 0,9\ \gamma_{f,E}$.

A.9.10.3.2.2 Design actions for extreme conditions when the action effects oppose

The partial action factors in Table 9.10-1 for extreme conditions are from the reliability calibration performed for the Gulf of Mexico structures[A.9.10-3]. The calibration considered wind, wave and current actions, permanent actions and the strengths of various structural components. The extreme environmental conditions exclude Q_2 ($\gamma_{f,Q2} = 0$) due to its short duration and small probability of occurring simultaneously with a storm.

The bottom line of Table 9.10-1 covers the situation of counteracting effects when two representative internal forces are similar in magnitude but opposite in sign. The algebraic sum of the forces is small which leads to a small required resistance. Due to the greater uncertainties in environmental actions, the safety index (β) can be low (high risk). This phenomenon is intensified when the structure's component strength is different for changes in the sign of the internal force (e.g. piles in tension instead of compression or axial members in compression instead of tension).

In the API LRFD calibration, counteracting effects were considered an important load case for uplift on piles. The reduced partial action factors when action effects from permanent and variable actions oppose those from environmental actions ensure adequate pile tension capacity. A reduced factor on G_1 implicitly covers cases of reduced topsides actions. The 0,8 on Q_1 is smaller than the 0,9 on G_2, reflecting its greater likelihood of not being the value assumed. However, reductions are even applied to the very certain and always present G_1, because it is the uncertainty in wind, wave and current that this check is intended to guard against. The safety index is thereby raised to a level consistent with the pile compression failure mode.

For brace members in the structure, an analogous case of counteracting effects exists. A calibration structure (Platform “C”)[A.9.10-3] was investigated for this phenomenon by looking at the most critical component in each design group. Although the partial action factors in Table 9.10-1 for extreme conditions when the action effects are additive serve to raise β compared to WSD for opposing load cases, it was deemed prudent to have the values for when action effects oppose apply to all structure components to guard against low β cases.

A.9.11 Local hydrodynamic actions

The Morison equation accounts for local drag and inertia actions but not for the “out-of-plane” (i.e. the plane formed by the velocity vector and the member axis) local actions caused by hydrodynamic lift due to periodic, asymmetric vortex shedding from the downstream side of a member. Hydrodynamic lift can be neglected in the calculation of global actions on the structure. Due to their high frequency, random phasing, and oscillatory (with zero mean) nature, lift actions are not correlated across the entire structure. However, such lift actions should be considered for local member design, particularly for members high in the structure whose internal forces can be dominated by locally generated actions.

The oscillating lift actions can be modelled as a modulated sine function, whose frequency is generally several times the frequency of the wave, and whose amplitude is modulated with U^2, where U is the time-varying component of fluid velocity normal to the member axis. In the absence of dynamic excitation, the maximum amplitude of the local lift action, $F_{L,max}$, per unit length of the member is related to the maximum value of U during the wave cycle (U_{max}), by Equation (A.9.11-1):

$$F_{L,max} = C_{L,max} \cdot 1/2\ \rho_w D U^2_{max} \tag{A.9.11-1}$$

The coefficient $C_{L,max}$ has been found empirically[A.9.11-1] to have considerable scatter, with an approximate mean value of $C_{L,max} \approx 0{,}7\ C_d$ for both smooth and rough circular cylinders, and in both steady flow and waves with large Keulegan-Carpenter numbers. Reference [A.9.11-2] focuses on the root-mean-square (rms) value of the oscillating lift load, finding that it was less than half $F_{L,max}$.

The frequency of the oscillating lift action is $Sr\ U_{total}/D$, where Sr is the Strouhal number and U_{total} is the total incident velocity, including the axial component. Laboratory tests[A.9.11-3],[A.9.11-1] have shown that $Sr \approx 0{,}2$ for circular cylinders over a broad range of Reynolds numbers and flow inclination angles in steady flow. If Sr remains constant in waves, the frequency of the oscillating action varies as U varies with time during a wave cycle.

In the event that any natural frequency of a member is near the frequency of lift actions, a large amplitude dynamic response called vortex induced vibration (VIV) can occur. When VIV occurs, the motion of the member and the magnitude of the fluid-dynamic actions can increase to unacceptable levels. VIV can occur on long spans due to actions caused by wind in the construction yard and on the tow barge, as well as by waves and currents on the in-place structure.

Horizontal members in the wave zone (between wave crest and trough elevations) of an in-place structure can experience actions due to wave slam. These nearly vertical actions are caused by the local water surface rising and impacting on the underside of the member as a wave passes. Since these actions are nearly vertical, they contribute very little to the base shear and overturning moment of the structure. However, actions due to slamming should be considered in local design of members in the wave zone.

Actions due to slamming can also occur on structural members overhanging a barge while the structure is being towed, or on members that strike the water first during side launching of structures.

In the theoretical case, slamming actions are impulsive. If the slamming action is truly impulsive, the member can be dynamically excited. In the real world, the slamming action is possibly not impulsive because of the three-dimensional shape of the sea surface, the compressibility of air trapped between the member and the sea surface and the aerated nature of water near the free surface.

Slamming action, F_s, per unit length may be calculated using Equation (A.9.11-2):

$$F_s = C_s \cdot 1/2\ \rho_w D U^2 \tag{A.9.11-2}$$

where U is the component of water particle velocity normal to the member axis at impact.

Reference [A.9.11-2] shows empirically that the coefficient, C_s, lies between 0,5 and 1,7 times its theoretical value of π, depending on the rise time and natural frequency of the elastically mounted cylinder in tests. Reference [A.9.11-4] recommends that if a dynamic response analysis is performed, the theoretical value of $C_s = \pi$ may be used; otherwise, a value of $C_s = 5{,}5$ should be used.

Additional information on slamming actions and dynamic response is provided in Reference [A.9.11-5].

Axial Froude-Krylov actions have the same form as the inertia action in Morison's equation, except that C_m is set to unity and the normal component of the local acceleration is replaced by the axial component. Such actions on members that are nearly vertical contribute negligibly to the structure's base shear and overturning moment. Axial Froude-Krylov actions on diagonal and horizontal braces are relatively more important, contributing approximately 10 % as much as the inertia action included in Morison's equation to base shear and overturning moment, based on computations in Reference [A.9.11-6]. In view of approximations made elsewhere in the computation of global environmental action, axial Froude-Krylov actions can generally be neglected.

A.10 Accidental situations

A.10.1 General

Consideration should in particular be given to accidental events arising from vessel collision and dropped objects. The severity and likelihood of these accidental events depend on many factors, including operating environment, service vessel size, material handling equipment, operator qualifications and proximity to shipping lanes. The specified accidental design situations and associated design criteria for collisions and dropped objects should therefore depend on the platform location, operational procedures adopted and safety measures taken.

Design criteria for collisions and dropped objects are difficult to quantify for three reasons:

— the interaction of human and equipment error that combine to cause an accidental event cannot easily be quantified in probabilistic terms;

— satisfying arbitrary or simplistic quantification rules is no guarantee that the required safety for a given accidental event will be achieved;

— blindly satisfying simple rules can exclude the option of implementing potentially inexpensive preventive measures derived from a more in-depth understanding of the problem.

Where the consequences of an accidental event are very great, and the probabilities of events occurring are very low, preventive measures can serve to provide the required safety. For example, measures to prevent collision by tanker are acceptable if they are effective. Designing to resist a tanker collision is not a practical approach.

A.10.2 Vessel collisions

A.10.2.1 General

No guidance is offered.

A.10.2.2 Collision events

The kinetic energy of a vessel may be calculated using Equation (A.10.2-1):

$$E = 1/2\ (a \times m)\ u^2 \qquad \text{(A.10.2-1)}$$

where

E is the kinetic energy of the vessel;

a is the added mass coefficient, with a = 1,4 for a broadside collision, a = 1,1 for a bow/stern collision;

m is the vessel mass;

u is the velocity of vessel at impact.

The added mass coefficients given above are typical values for large (5 000 t displacement) supply vessels. For smaller vessels, a value slightly higher than 1,4 should be applied, e.g. 1,6 for a typical 2 500 t supply vessel.

The representative velocity and size of the vessels used for impact analyses should correspond to those used in the operation and servicing of the platform (e.g. supply boats). By way of example, for the northern North Sea, a vessel mass can be 8 000 t, whereas in the southern North Sea a mass of around 2 500 t is more normal. For Gulf of Mexico structures in mild environments and reasonably close to their base of supply, a 1 000 t vessel represents a typical 55 m to 60 m (180 ft to 200 ft) supply vessel. For deeper and more remote locations in the Gulf of Mexico the vessel mass can be different. The masses of vessels that could collide with the platform when drifting out-of-control should be specifically considered.

The two energy levels specified for vessel impact analysis represent a frequent condition and a rare condition. The low energy level represents a serviceability condition based on economic considerations, and is intended to ensure that the structure will not require to be shut-in or need major repairs after minor collisions. The high energy level represents a rare condition and an ultimate limit state in which progressive collapse should not occur and the safety of personnel should be ensured, although the structure could be substantially damaged.

For low energy impacts, a vessel velocity of 0,5 m/s is commonly used, representing a minor accidental “bump” during normal manoeuvring of the vessel while loading or unloading or while standing alongside the platform. For high energy conditions, a vessel velocity of 2 m/s is commonly used, representing a vessel drifting out-of-control in a sea state with significant wave height of approximately 4 m.

A.10.2.3 Collision process

The majority of ship impact analyses performed to date have used quasi-static methods based on energy absorption. However, when the duration of the collision is short, dynamic effects can be significant.

The collision duration depends on the size and configuration of both the structure and the vessel, and on the nature of the collision. Dynamic effects can be significant when the duration is of the same order or less than the structure's natural period. In such cases an assessment of the dynamic behaviour during the collision should be considered.

Dynamic collision analyses can be carried out as time domain dynamic calculations in which the collision actions represent both the direct impact and the inertia of the structure. Because the two excitations do not attain their maximum value at the same time, the duration of the time simulation should be long enough to cover all relevant phases of the collision. Typically, the direct impact due to the collision attains its maximum value early during the collision, while the effects of inertia reach their maximum values later during the collision. Energy is absorbed in both the structure and the vessel, but additional absorption sources such as energy imparted to platform vibration and energy dissipating from radiating waves generated as a result of the collision can also be represented.

A.10.3 Dropped objects

No guidance is offered.

A.10.4 Fires and explosions

A number of documents address both hazard management in general (e.g. ISO 17776[A.10.4-1]) and more specific issues associated with fires and explosions (e.g. ISO 13702).

The industry associations have produced their own more detailed guidance applicable to particular types of operation and circumstances and these include API[A.10.4-2], which can be used for Gulf of Mexico type platforms, UKOOA[A.10.4-3, A.10.4-4, A.10.4-5, A.10.4-6], which are suited to larger platforms operated in a safety case regime, and NORSOK[A.10.4-7], which contains explicit analytical requirements.

A.10.5 Abnormal environmental actions

No guidance is offered.

A.11 Seismic design considerations

A.11.1 General

No guidance is offered.

A.11.2 Seismic design procedure

No guidance is offered.

A.11.3 Seismic reserve capacity factor

In both the simplified and detailed seismic action procedures, the ALE return period is determined so that the structure/foundation system is expected to meet the target annual probability of failure. The ELE return period is then determined from the ALE event so that a balance exists between the ELE and ALE requirements, i.e. that the ratio of ALE to ELE spectral accelerations is approximately equal to the inherent static and dynamic reserve strengths of the design. Having this balance, a fixed steel structure that is designed for ELE should have a high likelihood of meeting the ALE design demand so that costly design cycles can be avoided.

A good initial estimate for the seismic reserve capacity factor, C_r, will ensure that a balance exists between the ELE and ALE requirements. If the initial estimate of C_r is too high, the ALE check can lead to major design modifications and costly design cycles. On the other hand, if the initial estimate of C_r is too low, the design would be conservative and the ALE design check could be easily met.

Aside from having a balance between the ELE and ALE requirements, the guidelines also require that the ELE design should lead to an economically viable structure, i.e. the structure should not be severely damaged when subjected to more frequent earthquakes than the ALE. This economic objective is presented in terms of minimum ELE return periods which depend on the platform exposure level (L1, L2 or L3). Because this economic objective is not explicitly checked in the simplified seismic action procedure, and to avoid ELE return periods that are too low, the C_r factor is limited to 2,8 for L1 structures, 2,4 for L2 structures and 2,0 for L3 structures.

A.11.4 Recommendations for ductile design

The recommendations given in 11.4 are mainly based on experience of seismic design of offshore California (USA) structures and are consistent with Reference [A.9.1-1]. The industry has shown that these recommendations lead to structural/foundation designs that are ductile and have high seismic reserve capacity factors.

The legs of the structure should be designed against twice the design environmental action, E, during the ELE event. This is to ensure that braces fail before the legs and to avoid a premature collapse of the structure. For primary joints, it is desirable that braces achieve their full compressive or tensile strength before the joints fail. In lieu of the recommendation for designing joints in accordance with 14.2.3, the survival of joints against actions calculated in non-linear ALE time-history analyses may be demonstrated in accordance with 11.6.4. Using this procedure, joints should be checked for internal member forces at each time step of the time-history analysis.

A.11.5 ELE requirements

No guidance is offered.

A.11.6 ALE requirements

A.11.6.1 General

No guidance is offered.

A.11.6.2 ALE structural and foundation modelling

The foundation strength can have a large effect on the energy dissipation mechanisms within the structure-foundation system, and sensitivity to higher than expected soil stiffness should be investigated.

A.11.6.3 Non-linear static pushover analysis

Dynamic time-history techniques may also be used to effectively replicate static pushover analyses, see, for example, Reference [A.11.6-1].

A.11.6.4 Time-history analysis

No guidance is offered.

A.11.7 Topsides appurtenances and equipment

Due to the structure's dynamic response and amplification of the seismic accelerations, topsides design accelerations are typically much larger than those at the sea floor. The response spectra for different points on the topsides can vary significantly depending on the asymmetry of the design and the location of the centre of mass of the topsides.

In general, most properly anchored topsides appurtenances are sufficiently stiff that their lateral and vertical responses may be assumed to be the same as the maximum accelerations of their support points on the topsides. Drilling rigs, flare booms, deck cantilevers, tall vessels, large unbaffled tanks and cranes do not meet this rigid body criterion.

For topsides equipment, the partial action factor increase from 0,9 to 1,15 for strength level earthquake actions is intended to provide a margin of safety in lieu of performing an explicit ductility level analysis for those components.

Two methods of analysis for topsides equipment, piping and appurtenances are allowed: coupled and uncoupled analyses. A coupled analysis can result in more accurate, and often lower, design accelerations than those derived using uncoupled floor response spectra. A coupled analysis may be used when the effects of dynamic interaction are significant. Dynamic interaction becomes significant for more massive topsides components with natural periods that are close to the dominant modal periods of the structure. The variability in natural periods of components can have a significant impact on the reactions between components when performing coupled analyses and it can be necessary to perform sensitivity analyses to determine the range of the reactions due to modelling uncertainty. A coupled analysis is not necessary when the topsides equipment mass is less than 1 % of the global mass of the whole platform.

Equipment should be restrained by means of welded connections, anchor bolts, clamps, lateral bracing or other appropriate tie-downs. The ELE design of restraints should include both strength considerations as well as their ability to accommodate imposed displacements.

A.12 Structural modelling and analysis

A.12.1 Purpose of analysis

No guidance is offered.

A.12.2 Analysis principles

A.12.2.1 Extent of analysis

No guidance is offered.

A.12.2.2 Calculation methods

Hand and spreadsheet calculations are generally limited to simple components of the structure (beams, regular panels, etc.) under simple actions. The methodology used should reflect standard engineering practice with due consideration for the conditions of equilibrium and compatibility. Elastic or plastic design principles may be adopted, dependent on the limit state being checked.

Analysis of the structural system and detailed assessment of its components is normally performed by the finite element method. Finite element analysis (FEA) involves subdivision of the structure or a component into elements of known behaviour (e.g. beam, plane stress, plate bending, shell) and subsequent matrix solution on a computer. The method allows the behaviour of large and complex structures to be determined in terms of displacements, internal forces and stresses. Furthermore, once a computer model has been created, different types of analysis can be performed and repeated with ease.

Certain checks need to be undertaken for each analysis, whether by hand calculations or by computer, to ensure that gross errors have not been made and the results are sufficiently accurate. Advice on such checking, and also on the validation and verification of structural analysis software is given in Reference [A.12.2-1].

A.12.3 Modelling

A.12.3.1 General

No guidance is offered.

A.12.3.2 Level of accuracy

No guidance is offered.

A.12.3.3 Geometrical definition for framed structures

A.12.3.3.1 General

For steel space frame structures that respond statically, the influence of joint flexibility, offsets and eccentricities is discussed in A.12.3.3.1 to A.12.3.3.3 in terms of their effects on member forces and on member strengths. For dynamically sensitive structures, these factors can also affect the overall stiffness of the structure and hence the natural period. These aspects are also discussed in A.12.3.3.3.

Changes in dimensions as a result of design changes should be monitored during and after the completion of an analysis. Where these impact on the accuracy of the analysis, the changes should be incorporated and evaluated by reanalysis of the component or the complete structure. Ideally, the final analyses should reflect the as-built structure.

The secondary moments in the chord member of K- and X-joints are proportional to the product of the offset and the component of the axial brace forces that is perpendicular to the chord. The secondary moments may be neglected for joints where the diameter, D, of the chord is substantially greater than the diameters of the braces and where the offset is smaller than $D/4$. However, for joints where all members are of similar diameter, the secondary moments should be assessed. For a K-joint at the end of a chord member, the angle between one of the braces and the chord is approximately 90° (sometimes called an N-joint); in this case, the secondary moment cannot be distributed over the chord member on both sides of the joint, and the secondary moment may only be neglected if the offset is smaller than $D/8$.

A.12.3.3.2 Member modelling

The contribution of the horizontal plan members to the global lateral stiffness when the structure is in-place depends on the arrangement of the structure framing. The horizontal plan members do, however, contribute to the global torsional stiffness and also add stiffness during temporary phases, such as loadout/lift, transportation and launch. Furthermore, they often provide lateral support to conductors and other appurtenances. Hence, the main plan bracing members should be regarded as primary structure.

Where secondary members transfer or attract significant actions, they may be represented by equivalent members, with appropriate stiffness and properties, such that the applied actions are appropriately apportioned to the surrounding primary framework. For example, conductor guide frames within the plan framing, which directly support the conductors, may be considered as secondary framework and modelled by equivalent members. The simplification can be used to reduce the complexity of the computer model, the complexity of data input, the amount of output and the potential for errors. Care should, however, be taken to ensure that load cases (e.g. the wave crest positions) chosen for analysis are appropriate for the design of all classes of member, whether primary or secondary.

Modelling of structural components such as mudmats, shear plates and conductor guides can require finite element types other than beam elements (e.g. plane stress, plate bending, shell elements). Deck plating may be simulated in several ways, e.g. by truss (axial) elements to represent in-plane shear stiffness, by plane stress elements to represent both in-plane axial and shear stiffness, or by shell elements to represent membrane, shear and lateral bending stiffness.

The stiffness of appurtenances, such as launch rails (fitted to legs to provide a smooth launching), mudmats, J-tubes, risers and skirt pile guides, should be included in the model if they contribute significantly to the overall stiffness of the structure. It should be possible to model appurtenances without oversimplification, although care should be taken to ensure that very short stiff members do not cause numerical ill-conditioning of the global stiffness matrix.

A.12.3.3.3 Joint modelling

Joint flexibility does not significantly affect the primary axial forces in a simple framework [A.12.3-1]. However, joint flexibility does modify member end bending moments and mid-span moments.

In addition to modifying member bending moments, the inclusion of joint flexibility and member end offsets can reduce the overall stiffness of the framework and thus increase the main periods of vibration by 3 % - 6 % [A.12.3-1]. To avoid complex structural analysis, consideration of joint flexibility may be included by appropriate definition of the effective length (K) factors required to evaluate member strength, see 13.5. Similarly, the cross-section characteristics and the member end moments determine the moment reduction factor, C_m, for members subjected to combined axial compression and bending. Both K and C_m may be determined by separate rational analysis or prescribed conservative values in accordance with Table 13.5-1.

In a non-linear analysis to determine ultimate strength (reserve strength), joint flexibility should be considered and included, particularly if joint failure occurs before member failure and before the ultimate strength is reached. Joint failure will initiate the development of different internal force distributions.

A.12.3.4 Material properties

In linear elastic analysis the material properties required to determine stiffness are elastic (Young's) modulus, shear modulus and Poisson's ratio. To check the strength of members and joints, the specified minimum yield strength (SMYS) and ultimate tensile strength are required.

In non-linear analysis, in which internal forces and/or stresses are monitored as the analysis proceeds, the material properties should be defined by the appropriate “true” stress-strain curves incorporating first yield, strain hardening and tensile fracture.

For frameworks, non-linear material behaviour may be formulated using the plastic hinge theory based on a yield surface, plastic flow normal to the yield surface and an expanding yield surface (strain-hardening). The yield surface for a member is normally defined in terms of the axial yield strength, P_y, and the plastic moment strength about the principal axes of the cross-section, M_{Py} and M_{Pz}. Variables P_y, M_{Py} and M_{Pz} are all based on the yield strength from the “true” stress-strain curve.

For frameworks, tensile failure (fracture) of members and joints may be simulated by severing members.

A.12.3.5 Topsides structure modelling

Detailed requirements for the topsides structure are to be given in ISO 19901-3[2].

A.12.3.6 Appurtenances

In the past modelling of appurtenances during the design of a structure has often been ignored. Depending on their size and method of attachment to the structure, caissons and risers can provide a load path or can be subjected to action effects induced by relative movement between plan elevations arising from overall bending/sway of the structure. Although appurtenances, such as caissons and risers, may be regarded as secondary structures, they are often necessary for production or for safety reasons (e.g. fire water) and should therefore be given proper consideration.

A.12.3.7 Soil-structure interaction

A.12.3.7.1 General

Non-linear behaviour of a piled foundation arises from non-linear characteristics of the soil, second order axial force-displacement (P-Δ) effects and yielding in the pile. There are several ways of modelling single and group actions of piles, based on different implicit assumptions [A.12.3-2]. Their applicability depends on the importance of foundation characteristics in the overall response of the structure, the type of analysis (i.e. static in-place analysis, fatigue analysis, etc.) and the nature of the problem being studied (e.g. use of a detailed model for dealing with local effects).

To allow the soil to be modelled by a set of non-linear springs, simplifying assumptions are made regarding the stiffness, damping and added mass. Typically, it is assumed that the actions between the pile and soil at any level are independent of the deflections at any other level, that soil damping is part of the percentage critical damping assumed for the whole structure, and that the mass of soil acting with the piles has a negligible effect on the dynamic behaviour of the structure.

For well spaced piles interaction effects between piles need not be considered. However, for closely spaced piles, group behaviour should be considered, see A.12.3.7.2.

For approximate modelling of foundations and a preliminary estimate of pile behaviour, pile fixity may be assumed at a distance of 4 to 10 pile diameters below the sea floor (or depth of scour). Distances of 4 to 5 and 8 to 10 pile diameters are appropriate for stiff clays and very soft silts, respectively. Above this level of fixity, the pile is modelled using its actual cross-section properties and without lateral support (i.e. no soil springs).

The use of a simple foundation model is allowed for determining the behaviour of the structure — one such model consists of a beam fully fixed at its lower end and two lateral springs and one torsional spring at the sea floor. The properties of the equivalent system are generally chosen to represent the secant stiffness of the foundation. They are determined by comparing the pile (group) stiffness matrix, with interaction between lateral and rotational movement, and the stiffness matrix for the equivalent system. When used for fatigue analysis, a single linearization for all load cases due to wave actions that are expected to cause the most fatigue damage is usually acceptable, in the knowledge that for some cases it will be over-stiff and for other cases over-flexible, but that the “average” stiffness will be reasonable.

A full non-linear soil-structure interaction analysis combines a linear analysis of the structure with a non-linear analysis of the piled foundation. The non-linear soil p–y, t–z and Q–z curves are represented by piecewise linear springs and the piles by a linear or beam-column formulation. The solution involves substructuring of the structure to connection nodes at the pile heads, a non-linear iterative solution of the foundation and structure to determine the pile head displacements, and back-substitution of these displacements into the reduced equations of the structure to determine the displacements throughout the structure. Both the finite difference and finite element methods have been adopted in offshore structural analysis programs for analysing piled foundations.

Where an estimate of an existing structure's stiffness is available via natural frequency measurements, this should be used as a basis for the calculation of foundation stiffness; however, due account should be taken of the difference between the response of the foundation under normal and extreme situations. Any available driving records should also be reviewed for assessment of an existing structure.

A.12.3.7.2 Pile groups

For closely spaced piles, interaction effects between piles should be considered as their capacity can differ from the sum of the individual piles. Soil failure around the whole group should be prevented.

A pile group may be modelled by simpler means, such as an equivalent single member with equivalent structural and foundation properties. Normally, however, a detailed analysis of the structural components within the pile group is required.

A.12.3.7.3 Pile connectivity

No guidance is offered.

A.12.3.7.4 Conductor modelling

No guidance is offered.

A.12.3.7.5 Conductor connectivity

No guidance is offered.

A.12.3.8 Other support conditions

In all static analyses the structure is in equilibrium and should be prevented from rigid body movement. For launch and floating analyses, sufficient restraints should therefore be provided to suppress global motion in each of the six rigid body degrees of freedom (surge, sway, heave, roll, pitch and yaw) without inhibiting relative displacements within the structure. There will be no reactions at the restraints as long as the applied actions are balanced and the structure has the minimum (usually six) restraints required to avoid rigid body motion. The structure should be similarly restrained for lift situations. It is preferable to provide six translational restraints (two parallel to each axis and arranged to inhibit free rotation as well as free displacement, this requiring three non-collinear nodes that have some form of translational restraint) rather than to suppress all six freedoms (three translations and three rotations) at a single node. This minimizes the effect of any imbalance in the actions, since a minor error in global moment, while insignificant in overall terms, can be significant when applied as a reaction at a single point in the structure.

A.12.3.9 Local analysis structural models

Local analyses are structural analyses performed on a part or component of the overall structural system, their purpose being to provide a more accurate representation of the selected region than that provided by global analysis. They should typically provide more accurate information about the variation of section forces (or stresses) at complex regions of the structure. Local analyses may be non-linear in nature, where such effects are likely to include material yielding in areas of stress concentration at geometrical discontinuities.

The boundaries of a local analysis, and hence the extent of the local model, should be chosen so that displacements and/or forces from the model of the whole structural system (global analysis) can be readily applied as boundary conditions. The boundaries should be sufficiently far from areas of interest so that the forces and stresses in the areas of interest are not affected by any local effects at the boundaries due to the method of applying the boundary conditions or owing to any inaccuracies in the boundary conditions. For example, in a detailed finite element analysis of a tubular joint the braces should extend approximately two brace diameters to three brace diameters away from the heel of the brace-chord intersection to ensure that the stresses at the intersection are not affected by the boundary conditions at the end of the brace.

Displacements, forces and stresses at nodes or across a finite element's boundaries may be extracted from the global analysis in order to act as boundary conditions for the local analysis. Force boundary conditions should be used when the force distribution in the global analysis is considered to be correct.

The actions and boundary conditions applied to a local model should represent a self-equilibrating system. If the boundary conditions applied to a local model are extracted from an overall model, then the external actions applied to the idealization of the local model in the global analysis should also be applied to the local model (see A.12.3.10). Force and displacement restraints are both possible. Where it is uncertain which of these types of boundary condition is more appropriate, both should be used and the more onerous results retained.

Any significant actions within the boundaries of a local model should also be applied to the local model. This includes reactions due to support of the structure as well as other external actions. Such locally applied actions should be consistent with the corresponding actions in the global analysis from which the boundary conditions were obtained.

A.12.3.10 Actions

Crane lift conditions are significant for sub-assemblies and are often best modelled by simulating the slings as pin-ended ties fixed at the crane hook point or points. Imbalance between sling forces may be incorporated by replacing selected slings with reaction forces calculated to support a percentage of the total lift weight, or by specifying artificial temperature increases in the pin-ended ties representing the slings. In most cases additional fixity is required to restrain the structure against rigid body rotations about the hook point(s). It is usual practice to determine the centre of gravity of the structure and align this vertically below the hook point.

During transportation, the structure is supported by external water pressure, either directly applied or through a transportation barge. It is common practice to simulate this condition by applying the external water pressures as actions on the structure equal to and opposing all other actions (self weight, ballast, towing, etc.). The effects of acceleration (roll, pitch and heave) and tilt of the structure are usually taken into account by applying various combinations of mass, acceleration(s) (varying in magnitude and direction) and actions from self weights to a quasi-static model of the structure.

A.12.3.11 Mass simulation

In the finite element method, “consistent” element mass matrices are generated if the same interpolation functions are used as in the evaluation of the stiffness matrix. A “consistent” mass matrix has off-diagonal terms, indicating coupling between the various element degrees of freedom.

An approximate mass matrix may be established by “lumping” (distributing) an element's mass to the nodal points defining its topology. Thus the total mass at a nodal joint will represent the element-contributing volume around the node. The use of lumped (or diagonal) mass matrices can have a considerable computational advantage in the solution of the dynamic equations of equilibrium. For an assessment of the global response of framed structures, the use of lumped masses instead of consistent masses is fully acceptable, as this has only a minor effect on the results of a dynamic analysis. For other structural configurations, or for an assessment of the response of individual components, the influence can, however, be appreciably larger and a consistent mass matrix is recommended.

A.12.3.12 Damping

In a similar way to mass simulation (see A.12.3.11), “consistent” damping matrices can be derived, but in practice it is difficult, if not impossible, to determine the element damping parameters. Experimental results indicate that structural damping is frequency dependent, suggesting that damping is dependent on displacement amplitude. Models which take this into account are a hysteretic model, which is suitable for yielding materials, and a friction damping model which is suitable for some soils (e.g. sand) and joints where damping arises from friction between sliding surfaces [A.12.3-2].

Although hysteretic and friction damping are more representative of energy dissipation in an offshore structure, viscous damping is assumed for most analyses because it is mathematically more convenient and more important to model the correct energy loss per cycle than the exact mode(s) of damping. For damped vibration, viscous damping is usually expressed in terms of a damping factor, based on the ratio, $\xi = c/c_\mathrm{c}$, where c is a viscous damping coefficient and c_c is the critical damping. Critical damping is the amount of damping which separates the two types of response: over-damped decay without oscillation ($c/c_\mathrm{c} > 1{,}0$) and decaying sinusoidal response ($c/c_\mathrm{c} < 1{,}0$).

The choice of damping factor can have a profound effect on the predicted response. Values of 2 % of critical and less have been suggested on the basis of full-scale measurements in low sea states. For extreme sea states, a value of 5 % of critical is commonly used and for extreme seismic events, where inelastic behaviour of both structure and foundation is likely, a value of 10 % can be applicable, provided such higher value can be substantiated for the case under consideration. In dynamic analyses for a fatigue assessment, damping factors of 1 % to 2 % are appropriate; see Clause 16.

Including structural velocities in the calculation of hydrodynamic drag actions increases the predicted total system damping. For non-compliant structures this increase is included in the measured values and the damping based on relative velocities should not be used.

A.12.4 Analysis requirements

A.12.4.1 General

No guidance is offered.

A.12.4.2 Fabrication

No guidance is offered.

A.12.4.3 Other pre-service and removal situations

A structural component (or an appurtenance attached to the structure) can be subjected to large slamming actions if the longitudinal axis of the component is nearly parallel to the water surface as it enters the water. The slamming actions may be determined using the same procedures used to determine slamming actions due to waves.

A.12.4.4 In-place situations

A.12.4.4.1 General

Combinations of permanent and variable actions, particularly those involving drilling and production equipment that can be added to, removed from or repositioned on the structure, should be defined carefully so that each structural component will be assessed for the most onerous load case. Modern four-leg, vertical-pile structures, in particular, can experience net tension on the piles, and as the pile tension capacity is lower than its compression capacity, the tension case with minimum permanent and variable actions loading can control the pile design.

The design of legs and primary framing, particularly diagonal bracing in vertical frames, is generally governed by the actions causing maximum base shear or maximum overturning moment. However, primary and secondary structural components in horizontal frames, especially when these are located in the vicinity of the splash zone, are generally not governed by these wave positions; see also A.12.4.4.2.

A.12.4.4.2 Extreme environmental conditions

Internal forces in structural components are due to the force distribution associated with global actions as well as local actions on the component itself. It is often not obvious which combination of environmental parameters produces extreme global actions or what constitutes the most severe local situation for a particular structural component. Unless it is clear which wave directions and which crest positions govern the design of all structural components, it will generally be necessary to follow the following procedure.

a) Analyse eight wave directions corresponding to the main compass directions or the main directions relative to the structure. When there are significant differences in the environmental parameters from different directions it is usual to align the longer axis of a platform with the prevailing or most onerous direction. Symmetry in structure geometry and in environmental conditions from opposite directions can reduce the analytical requirements, but should be used with care as symmetry requirements are rarely fully satisfied.

b) Step each wave through the structure using a sufficient number of wave positions (phase angles) relative to the structure, to capture the governing positions, ensuring that governing positions for all structural components are identified.

Wave directions and wave positions should be selected by considering the environment (e.g. wave direction, wave length, wave height) as well as the geometry of the structure (e.g. leg spacing, elevation of plan framing levels).

A.12.4.4.3 Accidental situations

No guidance is offered.

A.12.4.4.4 Seismic events

No guidance is offered.

A.12.4.4.5 Fatigue analysis

No guidance is offered.

A.12.4.4.6 Analysis for reserve strength

A.12.4.4.6.1 Analytical software

Three basic types of analytical software are described below and are available for performing reserve strength analyses.

a) **General-purpose non-linear finite element programs**

   With these programs, the equations of equilibrium are usually evaluated in the deformed condition and, because the beam element formulations generally do not incorporate geometric non-linearity at the element level, multiple beam elements along a member span are required to model global member buckling response accurately.

b) **Non-linear beam-column space frame programs**

   These programs have specifically been written for the ultimate strength analysis of space frame structures. The elements embodied in this type of program have been derived to model beam–column behaviour, and therefore each member span only requires a single element to model buckling behaviour.

c) **Phenomenological space frame programs**

   In a phenomenological model, the failure of individual elements is prescribed through force–deformation relationships that can be empirically related to the element geometry. Non-linear analysis is less intensive given these relationships are known prior to the analysis. However, this typically requires some prediction of the type of member behaviour prior to the analysis, and can include the following behaviours.

   1) **Elastic members**

      These are members that are expected to perform elastically throughout the analysis.

   2) **Axial force members**

      These are members that are expected to undergo tensile yielding or axial buckling during ultimate strength analysis.

   3) **Moment resisting members**

      These are members that are expected to yield during an ultimate strength analysis, primarily due to high bending stresses. They should be modelled with beam-column elements that account for the formation of plastic hinges. A plastic hinge occurs when the axial and bending forces acting on a cross-section of the beam lie on the yield surface as defined by an axial force-bending moment (P/P_{y}, $M_{\mathrm{y}}/M_{\mathrm{Py}}$, $M_{\mathrm{z}}/M_{\mathrm{Pz}}$) interaction surface.

A.12.4.4.6.2 Structural failure modes

Care should be taken that the buckling response of beam elements that are used in analyses of type a), b) or c) according to A.12.4.4.6.1 include the possibility of localized failure (local buckling).

The following types of structural failure modes can often be identified. The modelling should be able to describe these failure modes.

a) Geometrically brittle failure modes, in which only compression members are involved in the failure mechanism (Z- and K-braced structures) and the collapse strength of the framework is achieved at first compression member failure. An accurate model of elasto-plastic buckling will therefore identify the structure's peak ultimate strength, which will occur at relatively small deflections, after which redistribution of internal forces occurs. As the lateral deflections increase, portal frame action occurs. Eventually plastic hinges develop in the legs and a full portal frame failure mechanism occurs in the bay containing the failed compression member. Prior to collapse, the forces in the members are well defined and therefore the ultimate strength depends on the ability to predict the member’s compressive strength, which should be reasonably accurate provided the yield strength is correct and fabrication tolerances for out-of-straightness have been considered.

b) Ductile failure modes, in which tension members are involved in the failure mechanism. Often, clear portal frame plastic failure mechanisms develop for which, at larger displacements when the internal forces in the compression member have been redistributed, the tensile member is the prime contributor to the collapse strength. In this state, the forces in all members of the failure mechanism are directly defined by equilibrium and the requirement to comply with plasticity limits. Again, it should be possible to predict the collapse strength and behaviour accurately.

c) Other modes of structural failure which are not as easily classified. In these failure modes, a progressive failure of members (and joints) occurs with a corresponding loss of global stiffness. At a certain level of actions, the global stiffness becomes zero and the ultimate strength is achieved. However, the deflections are still relatively small and a clear plastic portal frame failure mechanism has not yet developed. This is often seen in the failure mode for the diagonal wave attack direction of skirt pile structures; a corner leg in the bottom bay will fail in compression and the global stiffness will reduce to zero before a clear plastic failure mechanism develops.

The progressive collapse of a framework structure usually involves compressive and tensile member failures over complete spans. Localized failure (excluding brittle fracture) of a member's cross-section prior to global member failure does not occur often. Although joint failures can be viewed as localized failures, their failure prescribes the forces in and force redistribution from the attached members and hence can be effectively included in a component model of the members.

Material brittle failure modes should be avoided by specifying suitable steel toughness.

A.12.5 Types of analysis

A.12.5.1 Natural frequency analysis

When performing a natural frequency analysis solution techniques should enable all significant modes of vibration to be determined, i.e. both global and local modes of vibration. Methods involving the definition of a master set of nodes are generally not suitable because local modes are excluded when the stiffness and mass matrices for the complete structural system are condensed to the master nodes.

As predicted and measured natural periods of structures have sometimes been found to vary considerably, it is often prudent to shift the natural periods by as much as $\pm$ 10 % to a more onerous value. This may be accomplished by adjusting mass or stiffness within reasonable limits. Where measured periods are available (e.g. for assessment of existing structures) the model may be modified to reflect the measured values (e.g. by adjusting the elastic modulus or the foundation stiffness).

A.12.5.2 Dynamically responding structures

No guidance is offered.

A.12.5.3 Static and quasi-static linear analysis

The two methods for determining the correction for dynamic effects are applicable if the excitation causing the dynamic effects can be considered to vary approximately harmonically in time. If the variation in time is essentially different from harmonic the influence of dynamic effects should be specially considered.

Guidance on the two methods is given below.

a) The first method involves the determination of an equivalent set of quasi-static inertial actions that adequately represents the dynamic behaviour. When the dynamic response is caused by wave action this method is described in 9.8 and A.9.8 at a global analysis level for base shear and/or overturning moment. When the dynamic behaviour is due to another cause than wave action the equivalent quasi-static inertial actions corresponding with that cause can be determined in an analogous manner. After adding the equivalent actions to the real actions, both factored by appropriate partial action factors if required, one static analysis is performed. The action effects resulting from the static analysis are directly used in any further evaluation.

   This method is particularly recommended for parts of the structure that are above the application of the dynamic actions. These parts do normally not experience any quasi-static action effects, so when multiplied by a dynamic amplification factor [method b)] the quasi-static action effects will still be negligible, whereas the properly calculated dynamic action effects resulting from inertial actions can be significant.

b) The second method is performed in the following three steps.

   1) Determine the appropriate range of frequencies for which some influence of dynamic effects is expected to occur. For a number of discrete frequencies of excitation, ω_e, within this range a series of static analyses is performed. At each of these excitation frequencies the corresponding design actions are applied; note that the actions will in general be different for different frequencies. The calculated action effects of interest at each frequency are the static responses $r_{stat}(\omega_e)$; the static responses will in general also be a function of the frequency of excitation, because the actions are different for different frequencies.

   2) Determine the dynamic amplification factor (DAF), k_{DAF}, for a single degree of freedom system (SDOF) for each frequency of excitation, ω_e. The natural frequency of the SDOF system is the relevant natural frequency, ω_n, of the structural system, or structural part, that is responsible for the dynamic effect in the range of frequencies under consideration. The DAF is calculated using Equation (A.12.5-1):

$$k_{DAF}\left(\omega_e\right) = \frac{1}{\sqrt{\left(1-\Omega^2\right)^2 + \left(2\cdot\xi\cdot\Omega\right)^2}} \qquad \text{(A.12.5-1)}$$

   where

   Ω = ω_e/ω_n;

   ω_e is the frequency of excitation for which a static analysis is performed;

   ω_n is the relevant natural frequency of the structural system, or structural part, that is responsible for the dynamic effect;

   ξ is the fraction of critical damping associated with the dynamic effects (see A.12.3.12).

   k_{DAF} should not be smaller than 1,0, which corresponds to $\Omega \leqslant \sqrt{2}$.

3) Determine the dynamically amplified responses, $r_{\mathrm{dyn}}(\omega_{\mathrm{e}})$, from Equation (A.12.5-2):

$$r_{\mathrm{dyn}}(\omega_{\mathrm{e}}) = r_{\mathrm{stat}}(\omega_{\mathrm{e}})\, k_{\mathrm{DAF}}(\omega_{\mathrm{e}}) \tag{A.12.5-2}$$

The action effect that should be used in the further evaluation is the most onerous value of $r_{\mathrm{dyn}}(\omega_{\mathrm{e}})$ within the range of excitation frequencies considered.

A.12.5.4 Static ultimate strength analysis

The models used for describing tensile, compressive and bending ultimate strengths of tubular members and their cross-sections may be verified against the ultimate strengths derived from the formulae given in Clause 13, with all resistance factors set to 1,0. Differences can be checked to ensure that they fall within the range of expected scatter through statistics provided in Clause A.13. However, values greater than the mean of any distribution should be used with caution.

The post-buckling redistribution of forces, described by the axial force/axial shortening curve, should be verified to be kinematically correct, see References [A.12.5-1] and [A.12.5-2].

The model for combined compression and bending strength should, if possible, account for any reduction associated with local buckling of the cross-section. However, as local buckling does not usually occur in a well designed structure until after member failure, local buckling is unlikely to affect the ultimate strength of a structural system. Guidelines are provided in Reference [A.12.2-1], which also provides information on the increased rate of redistribution of forces associated with local buckling of the cross-section.

Models for compressive strength and redistribution of forces should take account of the effects resulting from transverse hydrodynamic member actions. Some non-linear beam-column and phenomenological space frame analysis programs only consider concentrated actions at the ends of members (often known as joint loads). This is because internal forces and moments in a primary member are predominantly caused by global frame behaviour and not by the hydrodynamic action imposed on the member itself. In these cases, local bending effects should be separately taken into account.

Hydrostatic pressure can reduce the strength of submerged buoyant members, and this should be taken into account in the modelling.

Phenomenological models for member behaviour generally require effective length (K) factors to be defined. The factors are used to establish the strength of members in compression in an ultimate strength assessment. Factors substantially lower than those specified in Clause 13 may be applied, if it can be demonstrated that they are both applicable and valid. It is suggested that, prior to performing pushover analyses, detailed elastic analysis be undertaken to determine accurate effective length factors; these factors are subsequently used in the pushover analyses. However, the effective length factors can require revision when joints begin to fail and there is a loss of member end restraint.

When creating structural models for a non-linear analysis, due consideration should be given to out-of-straightness, out-of-roundness and locked-in forces arising from the methods of fabrication (see Annex G).

Typical foundation failure modes to be checked include pile punch-through, pile pullout and pile lateral failure. These are failure mechanisms that are unlikely to be influenced by variations in the local flexibility/stiffness of the framework. It is therefore possible to capture each of these three failure modes using simple strength models, where detailed modelling of the foundation using beam-column elements for the pile and non-linear springs for the soil by p–y, t–z and Q–z curves is not required. A description of foundation failure modes is given below.

a) Pile punch-through or pile pullout

This occurs when the applied actions produce an overturning moment that results in a pile head axial force that exceeds the pile capacity in compression (punch-through) or tension (pullout). The capacity is determined by the skin friction strength given by t–z curves and, for pile compression, by end bearing strength given by Q–z curves, see Clause 17. The yield strength of the pile usually exceeds the pile capacity, and therefore full axial plasticity is unlikely to form in the piles. As piles pullout or punch through, force redistribution in the piles and/or the structure can occur, depending on whether the soil maintains or loses its strength.

This type of pile failure will occur if the steel framework is stronger than, or of similar strength to, the foundation. If stronger, only the piles will fail; if similar, a combined framework and foundation failure mechanism can occur with the component failures in the framework being influenced by redistribution of the forces in the piles.

b) Pile lateral failure

This occurs when the applied actions produce a shear force in a pile that cannot be resisted by the lateral p–y resistance of the upper layers of soil. This results in excessive lateral movement of the piles and, therefore, additional bending moments, which combine with the existing pile axial forces to cause yielding and plastic hinges in the piles.

This type of failure will occur if the steel framework is sufficiently stiff and strong against lateral movement and the upper layers of the soil are relatively weak. When the soil fails, a portal frame mechanism develops in the piles below the sea floor with plastic hinges occurring at the sea floor and at some distance below the sea floor. When the portal frame failure mechanism has developed, the structure in the failed area is effectively statically determinate as the forces in the piles are defined by their plastic capacity and equilibrium. Since most of the energy dissipated will be associated with plastic hinge deformation in the piles, it is unlikely that the joint flexibility and/or local variation in the stiffness of the framework will affect the pile lateral failure mechanism.

Variability in structural steel strength and the mechanical behaviour of steel structures is relatively well researched and can be rationally quantified, see References [A.12.5-3], [A.12.5-4] and [A.12.5-5]. The variability in behaviour of offshore piled foundations is significantly less well researched. Often, when reanalysing hurricane damaged structures, pushover analyses predict pile failures, although these have not been observed (see References [A.12.5-6] and [A.12.5-7]). Design values for axial and lateral pile performance appear to underpredict actual capacity and performance, and it appears that there is far more modelling and parameter uncertainty in the description of axial pile capacity and lateral pile performance than in the description of the steel structure's strength.

A.12.5.5 Dynamic linear analysis

For a specific wave direction, the total action on a structure caused by waves depends on the wave frequency and the structure's geometry, e.g. leg spacing. Consequently, for certain wave frequencies, the total wave action can approach zero. Should any one of these wave frequencies coincide with a natural frequency of the structure, the dynamic response would be artificially minimized. Care should therefore be taken when selecting waves for dynamic analysis to ensure that the calculation frequencies associated with artificially minimized wave actions do not coincide with the natural frequencies of the structure.

Dynamic analysis should not be limited to one wave direction. A minimum of three directions should be considered, i.e. end-on, broadside and diagonal wave directions.

A.12.5.6 Dynamic ultimate strength analysis

The guidance given in A.12.5.4 is equally applicable to dynamic ultimate strength analysis.

A.12.6 Non-linear analysis

A.12.6.1 General

Reference [A.12.2-1] gives guidance on non-linear analysis methods including representation of non-linear member and joint behaviour.

A.12.6.2 Geometry modelling

Most finite element formulations are based on polynomial functions which describe the deformation of an element in terms of the displacements and/or rotations at the nodes of the element. The same polynomial functions are often used to describe the element geometry in terms of its nodal coordinates. Increasing the order of the polynomial (linear, quadratic, cubic, etc.) generally improves the accuracy of the element so that fewer elements are required, for example, to model non-linear geometrical behaviour due to elastic buckling. However, sufficient elements should be used to simulate initial imperfections and the various modes of buckling, including buckling due to combined axial forces and bending moments and also lateral torsional buckling of open sections (e.g. I-beams) due to major axis bending. To model lateral torsional buckling, a minimum of two elements are usually required between each laterally restrained section of a member if the mode is essentially sinusoidal.

The requirements for finite element models are generally not applicable to phenomenological models.

The guidance in A.12.4.4.6 also applies to non-linear analysis.

A.12.6.3 Component strength

No guidance is offered.

A.12.6.4 Models for member strength

Member failure usually involves yielding of the material. In most finite element formulations, material behaviour is monitored at specific locations within an element; for beams, these would be the numerical integration points along the beam and various points over the cross-section, including the extreme fibres, at the integration points. The finite element mesh for critical members should therefore be defined so that there are integration points close to the positions of maximum internal member forces; otherwise, the member strength is likely to be overestimated.

The same mesh requirement is applicable to non-linear finite element space frame programs based on plastic hinge formulations when the internal forces are monitored at the integration points along a beam.

Special-purpose programs for framework analysis have been developed based on adaptive meshing techniques and also on phenomenological models (see A.12.4.4.6). Their performance is often more reliable than general purpose non-linear finite element programs. A thorough understanding of the theoretical background and limitations of a non-linear framework analysis should be gained before use.

A.12.6.5 Models for joint strength

Joints which can participate in the failure mechanism should be identified by an initial non-linear analysis assuming rigid joints and checking the joint strengths using the formulae given in Clause 14 with resistance factors set to 1,0. Joint non-linear behaviour may be modelled by uncoupled non-linear springs, individual springs being used for axial force, in-plane bending and out-of-plane bending, as these are the force components used to calculate joint strength. Infinitely stiff springs are usually adopted for shear and torsional joint stiffness.

Empirical formulae have been derived to describe the force-deflection and moment-rotation behaviour of joints to ultimate member forces. The formulae cover uncoupled axial force, in-plane bending moments and out-of-plane bending moments for simple joints. They do not allow for interaction between force and moment components, e.g. axial force and in-plane bending moments. They should therefore be used with caution and checks should be undertaken to ensure that joint strengths in Clause 14 are not exceeded. Checks should also be carried out to ensure that the behaviour is realistic and the interaction equation for joint strength is satisfied. For example, if the axial strength has been reached as defined by the axial force-deflection curve, the magnitude of the other internal forces should be small and not increase unless axial force is redistributed; also, if the chord strength of a joint has been reached, the brace forces should not increase. The effect of redistribution of internal forces on joint behaviour should also be carefully examined.

A.12.6.6 Ductility limits

No guidance is offered.

A.12.6.7 Yield strength of structural steel

The steel used in a fixed offshore structure can come from several different sources, e.g. from different steel mills, even if it is purchased from a single vendor. It is also possible for steel made to one specification to be sold as being to a different specification, provided that it meets the chemical and mechanical requirements. It cannot therefore be assumed that all the steel in a fixed offshore structure is from a single statistical population, without examination of the mill certificates and/or analysis of a statistically significant number of coupon tests taken from plates or casts used throughout the structure. Unless sufficient data is available for the particular structure being analysed, generic data for the steel grades in use should be used. In assessing the strength of particular structural components, it is acceptable for individual measured yield strengths to be used where coupons are taken from the point under consideration, or where it can be demonstrated that the test piece is taken from the same manufactured plate, pipe or steel cast being assessed.

It should be noted that national standards for tensile tests can specify differing strain rates. Care should be taken to correlate strain rates applied during extreme and accidental situations with test data.

A.12.6.8 Models for foundation strength

A.12.5.4 contains descriptions of foundation failure modes, their possible interaction with structural failure modes and associated modelling features. The most likely modes of failure are pile pullout (insufficient t–z soil strength), pile punch-through (insufficient t–z soil strength and/or Q–z end bearing strength) and pile failure due to excessive bending (weak p–y soil strength). Simplified models for representing foundation failure should include, as a minimum, these three modes of failure.

A.12.6.9 Investigating non-linear behaviour

No guidance is offered.

A.13 Strength of tubular members

A.13.1 General

The requirements have been developed specifically for circular tubular shapes that are typical of offshore structure construction. The types of tubulars covered include fabricated roll-bent tubulars with a longitudinal weld seam, hot-finished seamless pipes and ERW pipes that have undergone some form of post-weld heat treatment or normalization to relieve residual stresses. Relief of the residual stresses is necessary to remove the “rounded” stress-strain characteristic that frequently arises from the ERW form of manufacture.

The design requirements are tailored to tubulars of dimensions and material yield strengths typical of offshore structure members (f_y < 500 MPa and $D/t \leqslant 120$). Application of the requirements to thin tubulars with high D/t ratios (> 120) and high strength steels (f_y > 500 MPa) can lead to unconservative results. For $D/t > 120$, guidance is, for example, available in References [A.13.1-1] and [A.13.1-2].

The ratio of the specified yield strength to the ultimate tensile strength should not exceed 0,90. The representative value of the yield strength for design should be limited to the minimum of either the tested yield strength or 90 % of the ultimate strength when the actual (tested) yield-strength-to-ultimate-strength ratio exceeds 0,90.

Fabrication imperfections should conform to the following:

a) global out-of-straightness smaller than length/1 000;

b) global out-of-roundness, i.e. maximum difference between any two diameters measured at one cross-section, smaller than 1 % of the nominal diameter, independent of diameter and applicable to all D/t ratios;

c) for $D/t > 60$, local out-of-straightness not exceeding 0,2 % of the nominal diameter when measured using a gauge length of $4\sqrt{R\ t}$, where R is the radius.

A.13.2 Tubular members subjected to tension, compression, bending, shear or hydrostatic pressure

A.13.2.1 General

The compression, bending, and hydrostatic pressure requirements contained in this subclause are based on tests carried out since 1975.

The screened test data used in the substantiation comparisons presented in A.13.2.3 to A.13.2.6 were collected from References [A.13.2-1] and [A.13.2-2]. Both of these references presented substantial collections of data, including both acceptable and non-acceptable test results. The main criterion for acceptability was completeness of data. This extended to include yield strength measurements for each specimen as well as individual values of critical geometrical variables such as diameter and thickness. The absence of measurements of elastic modulus alone was not considered reason enough to reject the data. Where elastic modulus was not measured, a value of 205,000 MPa was used when comparing measured and predicted strength. The lack of detailed measurements of critical geometry was usually taken as a reason to ignore data pertaining to columns, beam-columns and tubulars subjected to external pressure either alone or in combination.

Tubulars with initial geometry outside API RP 2B tolerances [A.13.2-3] were always rejected except those subjected to external pressure. Correction for out-of-tolerance construction was effected using the expression given in Reference [A.13.2-1] explicitly for this purpose. Column results that exhibited strengths in excess of the Euler buckling strength were rejected along with other specimens in the same series. This excess is only possible if the end fixities do not approach the simply supported end conditions claimed to exist during the test.

Static values of yield strength were used where available, dynamic values were used in their absence. In the form used for normalization of test results, dynamic yield strength values produced conservative points for comparison.

Specimens constructed from steel of thickness of about 1,8 mm and less were rejected because of unease over the potential for being out-of-tolerance and possibly of higher than usual variability in material properties. Most of the specimens in question belonged to early test series when the need and ability to accurately measure circularity was not always appreciated or possible. The possible variability in material properties was judged a consequence of the production processes available at the time. The need to take account of such issues to some extent reflected the inherent variability in the test results in questions.

Figures showing comparisons between test results and predictions using the equations given in 13.2 to 13.4 in support of the requirements of this International Standard are included in the corresponding subclauses of this annex. Also included in the figures are the statistics (such as the bias and COV) of the fit.

A.13.2.2 Axial tension

The equation for checking axial tension is applicable only for yielding of the gross section of circular tubulars. This condition should cover the large majority of tubular members in offshore structures.

The consequences of tensile yielding are less severe than other tubular member failure conditions, such as local buckling and beam-column buckling.

A.13.2.3 Axial compression

A.13.2.3.1 General

Tubular members subjected to axial compression can fail due to material yield, overall column buckling, local buckling, or combinations thereof. Column buckling is discussed in A.13.2.3.2, while local buckling and material yield are discussed in A.13.2.3.3.

A.13.2.3.2 Column buckling

The equation for the representative column buckling strength is a function of λ, a normalized form of column slenderness parameter given by $(f_{yc}/f_e)^{0,5}$ where f_{yc} is the local buckling strength of the cross-section (see A.13.2.3.3) and f_e is the Euler buckling strength for a perfect column. The equation is similar in format to the existing API LRFD equation [A.13.2-4]. Equations given in this subclause are appropriate for tubular column buckling accounting for fabrication imperfections. The buckling strength equates to 90 % of the Euler strength at large slenderness.

The column test database consists of 12 tests on fabricated tubulars, two on seamless pipe and 70 on ERW pipe, see References [A.13.2-5] to [A.13.2-8]. This database is considerably larger than that previously adopted for the purpose of substantiating offshore tubular strength formulations. The increase in the number of tests is primarily due to the inclusion of the relevant results from a large CIDECT test programme [A.13.2-8], although in the process of selection based on the criteria cited in A.13.2.1, a number of previously accepted data were rejected.

A comparison between the test data and predictions by the equations is shown in Figure A.13.2-1, together with the statistics of the fit. The representative column strength equation can be seen to approximate a lower bound of the tested strength, although it is not possible to verify this in the range $\lambda > 1{,}0$ where no relevant data exist. The overall fit achieved is good, with a bias of 1,057 and a COV of 0,041.

For a member composed of two or more different cross-sections along its length, the following steps can be used to determine the representative axial compressive strength, in force terms.

Determine the elastic buckling strength, P_e (in force units), for the complete member, taking into account the end restraints and variable cross-sectional properties. In most cases, the effective length factor for the member needs to be determined. The resulting representative axial compressive strength, $P_{c,r}$ (in force units), is then given by Equations (A.13.2-1) and (A.13.2-2):

$$P_{c,r} = \left(1 - 0{,}278\frac{P_{yc,r}}{P_e}\right)P_{yc,r} \qquad \text{for } \left(\frac{P_{yc,r}}{P_e}\right)^{0,5} \leqslant 1{,}34 \qquad \text{(A.13.2-1)}$$

$$P_{c,r} = 0{,}9\,P_e \qquad \text{for } \left(\frac{P_{yc,r}}{P_e}\right)^{0,5} > 1{,}34 \qquad \text{(A.13.2-2)}$$

where

$P_{c,r}$ is the representative axial compressive strength, in force units;

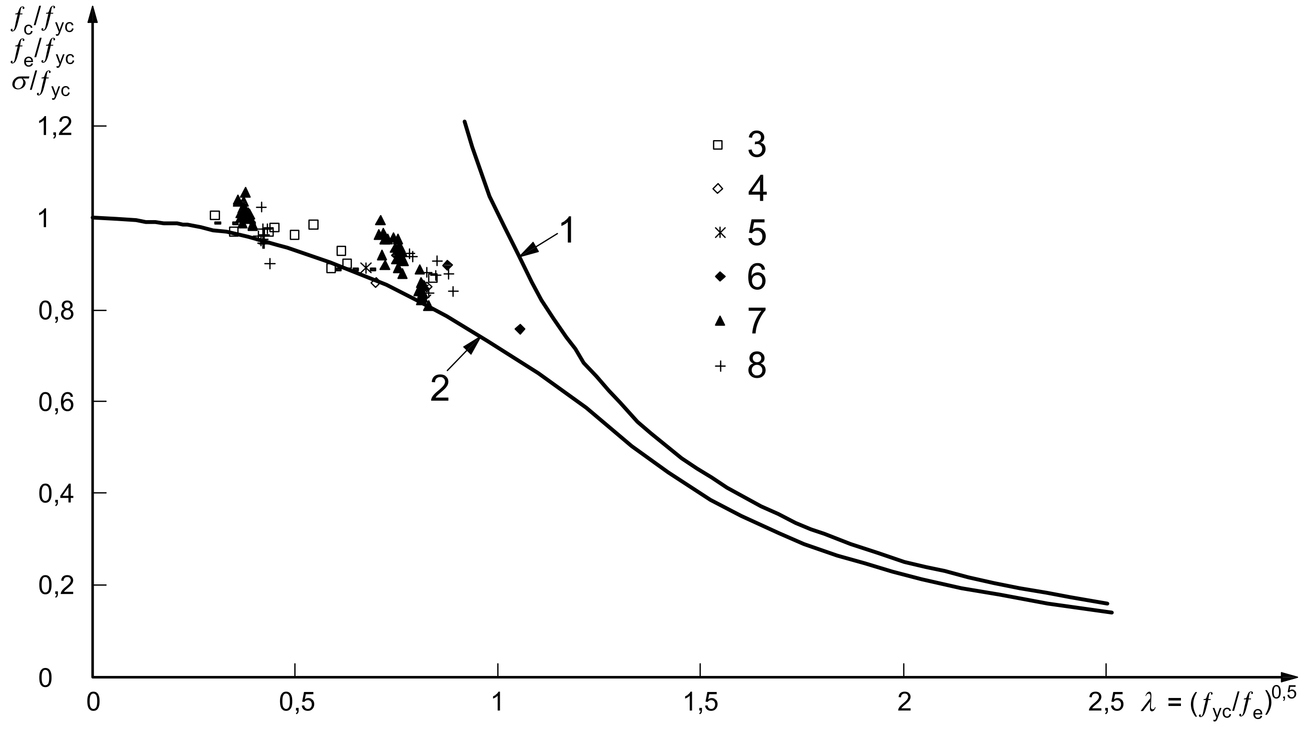
P_e is the elastic buckling strength of the complete member, in force units;

$P_{yc,r}$ is the smallest axial compressive strength of all the cross-sections, in force units, and is given as the minimum of $f_{yc,i}\,A_i$;

$f_{yc,i}$ is the representative local buckling strength as given by Equations (13.2-8) and (13.2-9) for the i^{th} section;

A_i is the cross-sectional area of the i^{th} section.

The representative axial compressive stress for each section is obtained by dividing $P_{c,r}$ by each of the respective cross-sectional areas, A_i.



Bias = 1,057 COV = 0,041 n = 84

Key

1 Euler buckling strength, f_e/f_{yc}

2 equations in this International Standard, f_c/f_{yc}

σ compressive stress in test member

f_c representative axial compressive strength

f_e Euler buckling strength

λ column slenderness parameter

f_{yc} representative local buckling strength

n frequency (number of occurrences)

Test data

3 Chen & Ross [a] [A.13.2-5]

4 Smith, Somerfield & Swan [a] [A.13.2-6]

5 Smith, Somerfield & Swan [b] [A.13.2-6]

6 Steinmann & Vojta [c] [A.13.2-7]

7 Yeomans [c] [A.13.2-8]

8 Yeomans [b] [A.13.2-8]

[a] Fabricated pipe.

[b] Seamless pipe.

[c] ERW pipe.

Figure A.13.2-1 — Comparison of test data with representative column buckling strength equations for fabricated cylinders subjected to axial compression

A.13.2.3.3 Local buckling

Short tubular members subjected to axial compression will fail either by material yield or by local buckling depending on the diameter to thickness (D/t) ratio. Tubular members with low D/t ratios are generally not subject to local buckling under axial compression and can be designed on the basis of material failure, i.e. the local buckling stress may be considered to be equal to the yield strength. However, as the D/t ratio increases, the elastic local buckling strength decreases, and the tubular should be checked for local buckling.

Unstiffened thin-wall tubulars subjected to axial compression and bending are prone to sudden failures at stress levels well below the theoretical elastic critical buckling stresses predicted by classical small-deflection shell theory. There is a sudden drop in load-carrying capacity upon buckling. The post-buckling reserve strength of tubular members is small, in contrast to the post-buckling behaviour of flat plates in compression, which usually continue to carry substantial load after local buckling. For this reason, there is a need for more conservatism in the definition of the buckling strength for tubulars than for most other structural elements. This is made difficult by the large scatter in test data, and necessitates a relatively conservative design procedure. The large scatter in test data is primarily the result of initial imperfections generated by fabrication tolerances and procedures. In addition to geometric imperfections, experimental and theoretical evidence has shown that the buckling strength is also affected by boundary conditions and built-in residual stresses, see References [A.13.2-5] and [A.13.2-9].

In order to achieve a robust design, member geometry should be selected such that local buckling due to axial forces is avoided.

The equations for the representative local buckling strength were developed by screening test data and establishing a strength curve at 95 % success at the 50 % confidence level, which satisfies the following conditions:

a) it had a plateau equal to the material representative yield strength over the range $0 < f_y/f_{xe} \leqslant 0{,}17$;

b) it had the general form of Equation (13.2-9);

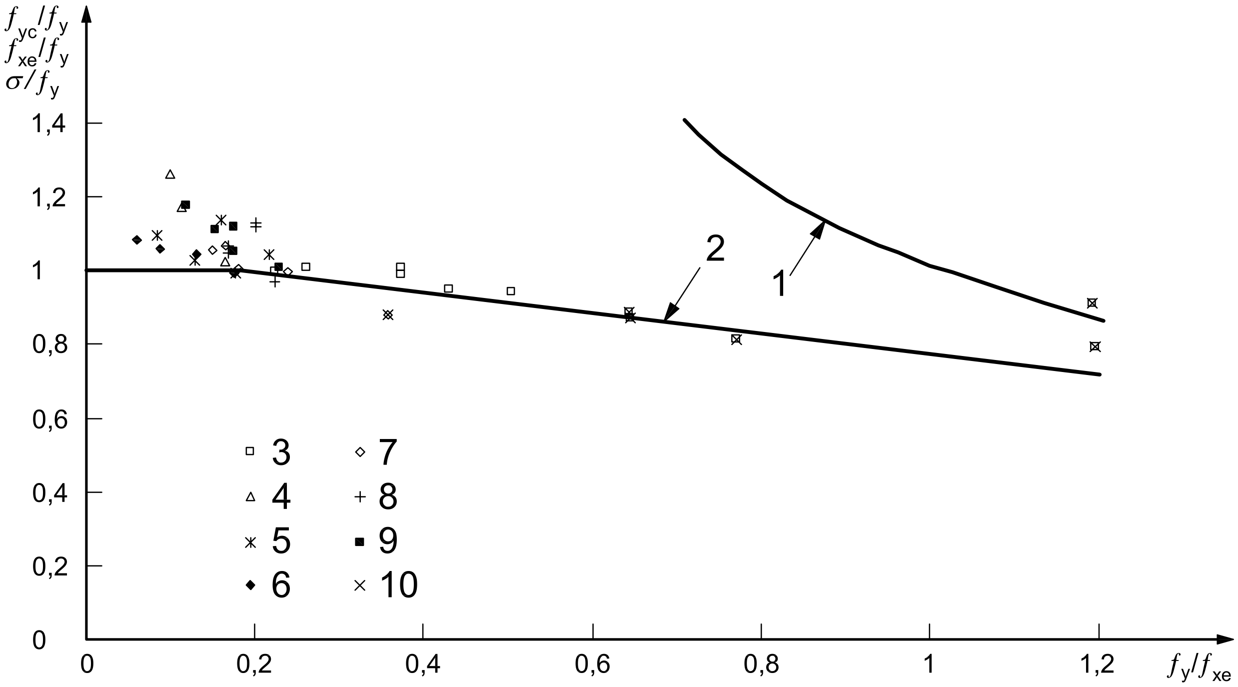
c) it converged to the elastic critical buckling stress curve with increasing member slenderness ratio;

d) the difference between the mean minus 1,645 standard deviations of test data and the developed equations was minimal.

The local buckling database consists of 38 acceptable tests performed by several different investigators, see References [A.13.2-5] and [A.13.2-10] to [A.13.2-14].

A comparison between test data and the representative local buckling strength equations [Equations (13.2-8) and (13.2-9)] is plotted in Figure A.13.2-2. The developed equations have a bias of 1,065, a standard deviation of 0,073, with a coefficient of variation of 0,068.

The elastic local buckling stress formula of Equation (13.2-10) represents one-half of the theoretical local buckling stress computed using classical small-deflection theory and is created by setting the buckling coefficient C_x to 50 % of its theoretical value, i.e. 0,3 instead of 0,6. This reduction accounts for the prime detrimental effect of geometric imperfections as well as the secondary effects of residual stresses and end conditions. Based on the test data shown in Figure A.13.2-2, it is considered to be conservative for tubulars with $t \geqslant 6$ mm and $D/t \leqslant 120$. Offshore structure members typically fall within these dimensional limits. For thinner tubulars and tubulars with higher D/t ratios, larger imperfection reduction factors can be required. For the design of tubular members beyond these dimensional limits, References [A.13.1-2] and [A.13.2-15] provide guidance.



Bias = 1,065 COV = 0,068 n = 38

Key

1 elastic local buckling strength, f_{xe}/f_y
2 equations in this International Standard, f_{yc}/f_y
σ compressive stress in test member
f_y representative yield strength
f_{yc} representative local buckling strength
f_{xe} representative elastic local buckling strength
D/t diameter to thickness ratio
n frequency (number of occurrences)

Test data

3 Marzullo & Ostapenko [A.13.2-10]
4 Chen & Ross [A.13.2-5]
5 Eder et al [a] [A.13.2-13]
6 Kiziltug et al [A.13.2-14]
7 Ostapenko & Grimm [A.13.2-11]
8 Prion & Birkemoe [A.13.2-12]
9 Eder et al [b] [A.13.2-13]
10 $D/t > 170$

[a] Fabricated pipe.
[b] ERW pipe.

Figure A.13.2-2 — Comparison of test data and the representative local buckling strength equations for cylinders subjected to axial compression

A.13.2.4 Bending

The representative bending strength of fabricated tubular members, as given in Equations (13.2-13) to (13.2-15), is obtained by dividing the (ultimate) plastic moment strength by the calculated elastic yield moment. A comparison of the representative bending strength with test data is shown in Figure A.13.2-3. The bias and COV of the strength equations are also given. The test data were taken from References [A.13.2-7], [A.13.2-14] and [A.13.2-16] to [A.13.2-19].

The test data show that simply-supported beam tests have bending strengths lower than corresponding fixed-end beam tests in which cross-sectional distortion is prevented. However, the end condition has little influence on the rotational capacity of a beam.

At low $\frac{f_y D}{Et}$, the plastic hinge mechanism forms over a short length of the beam. As the end support rigidity is reduced, the hinge mechanism forms over a longer length with the same rotation.

As $\frac{f_y D}{Et}$ increases, both the bending strength and the rotational capacity decrease.

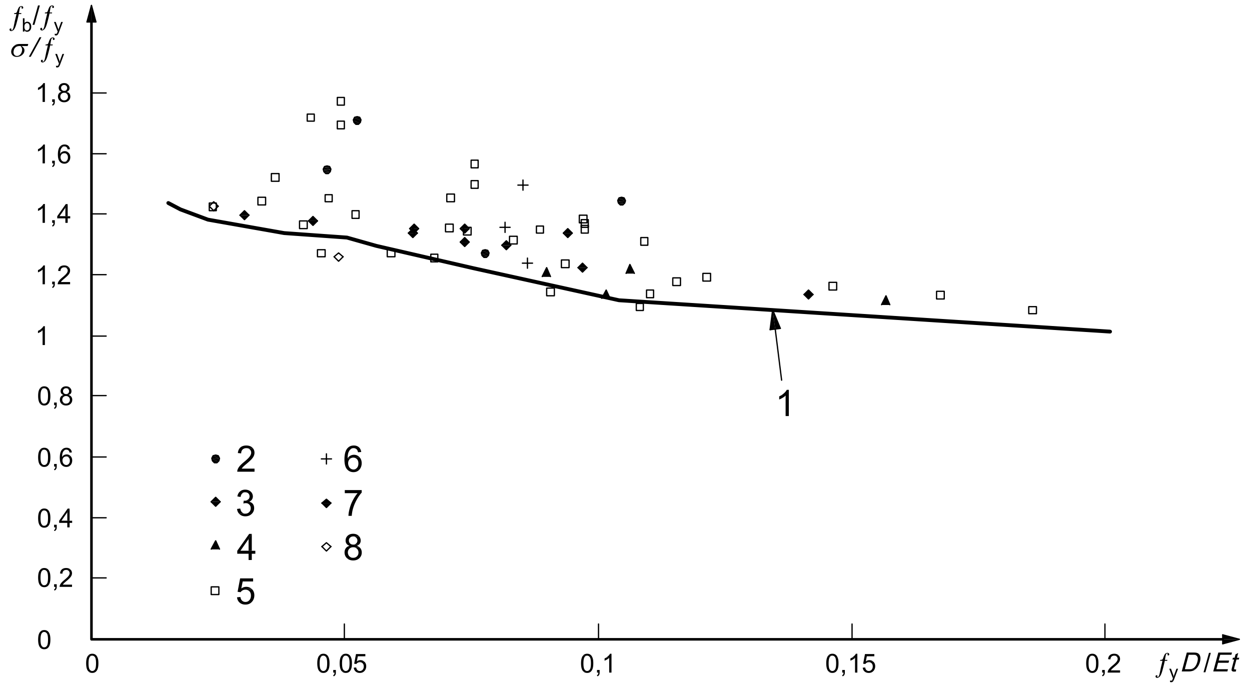
The behaviour of a tubular member under bending is grouped into three regions:

a) high rotational capacity — ductile failure mode, exhibiting very gradual strength decay, see Equation (13.2-13);

b) intermediate rotational capacity — semi-ductile failure mode, exhibiting gradual strength decay, see Equation (13.2-14);

c) low rotational capacity — little post-ultimate ductility, exhibiting rapid strength decay and susceptibility to pre-yield local buckling, see Equation (13.2-15).

The ultimate bending moment strength is the full plastic moment of the member. For members with $f_y = 345$ MPa and $E = 205\ 000$ MPa, the development of the full plastic moment is realized when $D/t \leqslant 30$, as reflected in Equation (13.2-13). This strength is linearly reduced to about 10 % in excess of the yield strength when $D/t \approx 60$ as reflected in Equation (13.2-14). Although it is not reflected in Equation (13.2-15), the bending strength of a tubular member with very high D/t should approach the local elastic critical buckling strength for a tubular member subjected to axial compression.



Bias = 1,109 COV = 0,085 n = 57

Key

1 equations in this International Standard

σ bending stress in test member

f_b representative bending strength

f_y representative yield strength

n frequency (number of occurrences)

Test data

2 Kiziltug et al [a] [A.13.2-14]

3 Korol & Hudoba [a] [A.13.2-17]

4 Sherman (1984) [a] [A.13.2-19]

5 Sherman (1987) [b] [A.13.2-16]

6 Steinmann & Vojta [a] [A.13.2-7]

7 Korol [a] [A.13.2-18]

8 Sherman (1984) [c] [A.13.2-19]

[a] ERW pipe.

[b] Fabricated pipe.

[c] Seamless pipe.

Figure A.13.2-3 — Comparison of test data with representative bending strength equations for fabricated cylinders

A.13.2.5 Shear

No guidance is offered.

A.13.2.6 Hydrostatic pressure

A.13.2.6.1 Calculation of hydrostatic pressure

No guidance is offered.

A.13.2.6.2 Hoop buckling

Unstiffened tubulars subjected to external hydrostatic pressure are susceptible to elastic or inelastic local buckling of the shell wall between restraints. Once initiated, the collapse will tend to flatten the member from one end to the other. Ring-stiffened tubulars are subject to local buckling of the shell wall between rings, with the rings remaining essentially circular. However, the rings can rotate or warp out of their plane. Ring-stiffened tubular members are also subject to general instability, which occurs when the rings and shell wall buckle simultaneously at the critical stress. It is highly desirable to provide rings with sufficient reserve of strength to prevent general instability.

For tubulars satisfying the maximum out-of-roundness tolerance of 1 %, the representative hoop buckling strength is given by Equations (13.2-23) to (13.2-25). For ring-stiffened tubulars these equations give the hoop buckling strength of the shell wall between rings. To account for the 1 % out-of roundness, the elastic hoop buckling stress is taken as 0,8 times the theoretical value calculated using classical small deflection theory, that is, $C_h = 0{,}44\ t/D$, whereas the theoretical value is $0{,}55\ t/D$. In addition, the other C_h values are lower bound estimates.

For members with an out-of-roundness greater than 1 %, but less than 3 %, a reduced elastic hoop buckling stress, $f_{he,red}$, is applicable [A.13.2-20]. The representative hoop buckling strength (f_h) is then determined using $f_{he,red}$.

$$f_{he,red} = \alpha f_{he}/0{,}8$$

where

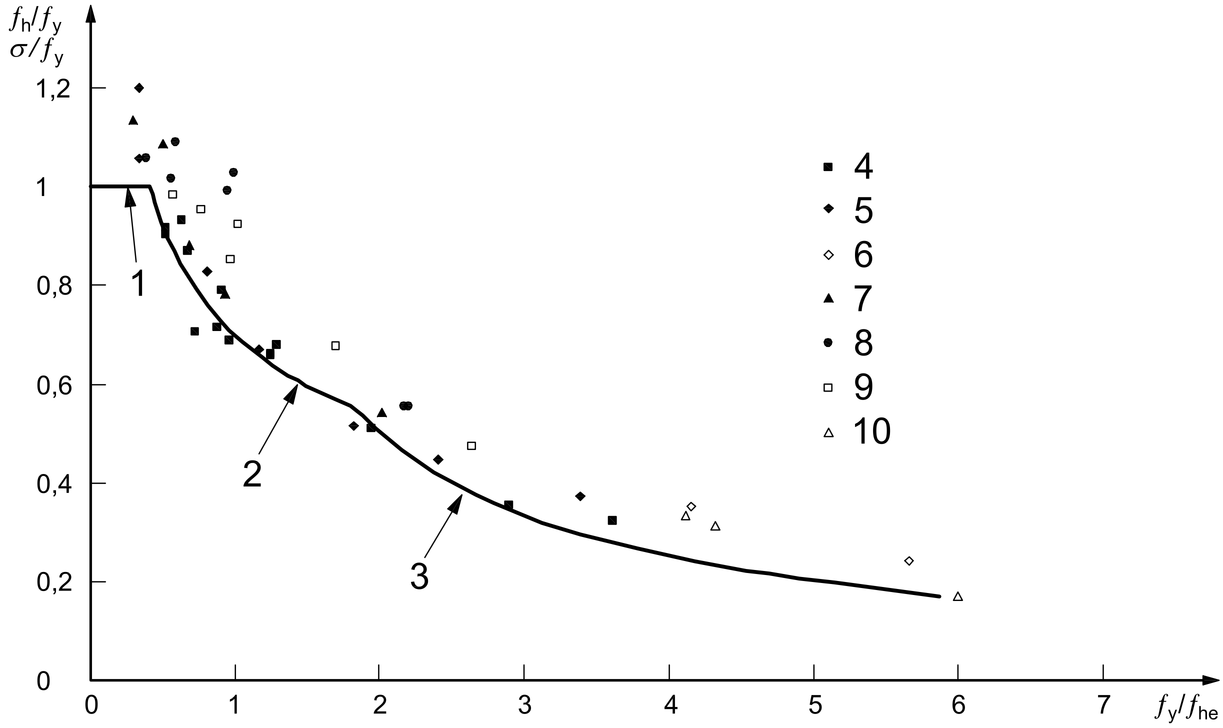
α is a geometric imperfection factor;

$$\alpha = 1 - 0{,}2\sqrt{\frac{D_{max} - D_{min}}{0{,}01\ D_{nom}}}$$

$\dfrac{D_{max} - D_{min}}{0{,}01\ D_{nom}}$ is the out-of-roundness (%)

where D_{max} and D_{min} are the maximum and minimum values of any measured outside diameter at a cross-section and D_{nom} is the nominal diameter. f_{he} is taken from Equation (13.2-26).

A comparison of the representative hoop buckling strength with test data is shown in Figure A.13.2-4. The bias and COV of the strength equation are also given. The test data were taken from References [A.13.2-20], [A.13.2-21], [A.13.2-13], [A.13.2-14] and [A.13.2-7].



Bias = 1,142 COV = 0,124 n = 44

Key

1 Equation (13.2-23), f_h/f_y
2 Equation (13.2-24), f_h/f_y
3 Equation (13.2-25), f_h/f_y
σ hoop stress in test member
f_h representative hoop buckling strength
f_{he} representative elastic critical hoop buckling strength
f_y representative yield strength
n frequency (number of occurrences)

Test data

4 Miller & Kinra [a] [A.13.2-21]
5 Miller, Kinra & Marlow [a] [A.13.2-20]
6 Miller, Kinra & Marlow [b] [A.13.2-20]
7 Eder et al [a] [A.13.2-13]
8 Eder et al [c] [A.13.2-13]
9 Kiziltug et al [d] [A.13.2-14]
10 Steinmann & Vojta [d] [A.13.2-7]

[a] Fabricated pipe with ring stiffeners.
[b] Fabricated pipe unstiffened.
[c] ERW pipe with ring stiffeners.
[d] ERW pipe unstiffened.

Figure A.13.2-4 — Comparison of test data with representative hoop buckling strength equations for fabricated cylinders subjected to hydrostatic pressure

A.13.2.6.3 Ring stiffener design

The formulae for determining the required moment of inertia of the stiffening rings, Equations (13.2-33) and (13.2-34), provide sufficient strength to resist buckling of the ring and shell even after the shell has buckled between stiffeners. It is assumed that the shell offers no support after buckling and transfers all its force to the effective stiffener section. The stiffener ring is designed as an isolated ring that buckles into two waves ($n = 2$) at a collapse pressure 20 % higher than the strength of the shell.

The dimensional requirements recommended in this subclause are based on an evaluation of References [A.13.2-22] to [A.13.2-24], and [A.13.2-15]. Table A.13.2-1 presents a comparison of the geometry requirements for stiffeners with flanges. The selected values are the same as those for Eurocode 3 [A.13.2-23], but the proposed expressions are presented non-dimensionally (they are also the same as those used for class I in NS 3472 [A.13.2-24], as required for the plastic design of beams subjected to compression).

Table A.13.2-1 — Comparison of geometry requirements in different codes for stiffeners with flanges when supported along two edges

Reference	b/t_f	h/t_w
AISC [A.13.2-22]	16,6	32
EUROCODE 3 [A.13.2-23]	13	25
NPD [A.13.2-15]	10	20
This International Standard	13,4	24,5
The data in the table apply for $E = 210\ 000$ MPa and a yield strength of 420 MPa.		

A.13.3 Tubular members subjected to combined forces without hydrostatic pressure

A.13.3.1 General

This subclause describes the background of the design requirements given in 13.3, which cover tubular members subjected to combined axial and bending forces without hydrostatic pressure.

In this subclause, as for 13.4, the designer should include second-order frame moments, i.e. P-Δ effects in the bending stress determination, when they are significant. The P-Δ effects could be significant in the design of unbraced deck legs, piles, and laterally flexible structures.

A.13.3.2 Axial tension and bending

No guidance is offered.

A.13.3.3 Axial compression and bending

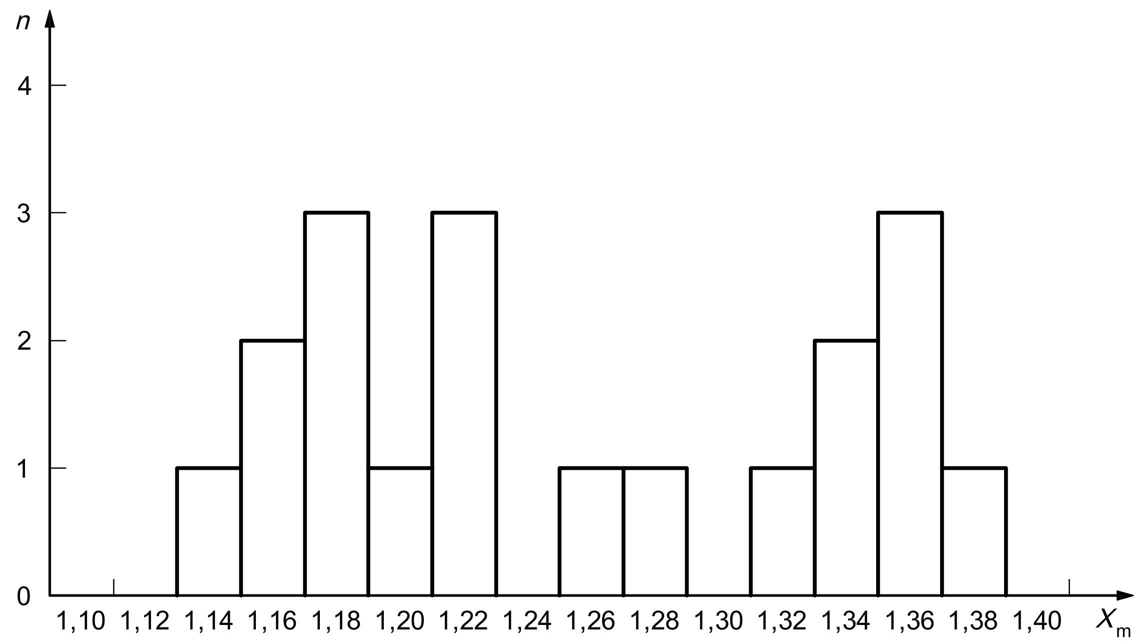
This subclause provides an overall beam-column stability check, Equation (13.3-3), and a strength check, Equation (13.3-4), for components subjected to combined axial compression and bending.

Equation (13.3-3) maintains the AISC-ASD beam-column stability interaction equation [A.13.2-22]. This equation is conservative with respect to small scale tubular member tests and analyses of larger members performed with assumed imperfections and residual stresses. As data accumulate on this subject, a reliability calibration of the form of this interaction will be appropriate.

Equation (13.3-4), the interaction equation for the strength check, is of the "linear form" which is simpler and slightly conservative for $D/t < 25$ compared with that of the "cosine form". Thus, comparisons for members with dominant bending can show higher interaction ratios. The influence of D/t on the strength interaction equation is contained in both f_{yc} in Equations (13.2-8) to (13.2-10), and in f_b in Equations (13.2-13) to (13.2-15). For low D/t ratios (< 25) there is some justification for using a "cosine" interaction equation. However, the test evidence for this is limited and much of it relates to short specimens ($L/D = 3$), so there does not appear to be strong support for its use at this time. To use the cosine form of interaction equation for low D/t ratios (< 25), the appropriateness needs to be demonstrated on each occasion.

NOTE The ISO equation for axial compression and bending is identical to that used in Reference [A.13.3-1].

The test data for tubular members subjected to combined axial compression and bending were taken from References [A.13.2-9], [A.13.2-12], [A.13.2-14] and [A.13.3-2] to [A.13.3-4]. The comparisons between the test data and the predicted strength of tubulars subjected to combined axial compression and bending are plotted in Figures A.13.3-1 and A.13.3-2, for short and long columns, respectively. The statistics of the fits are also included in the figures.



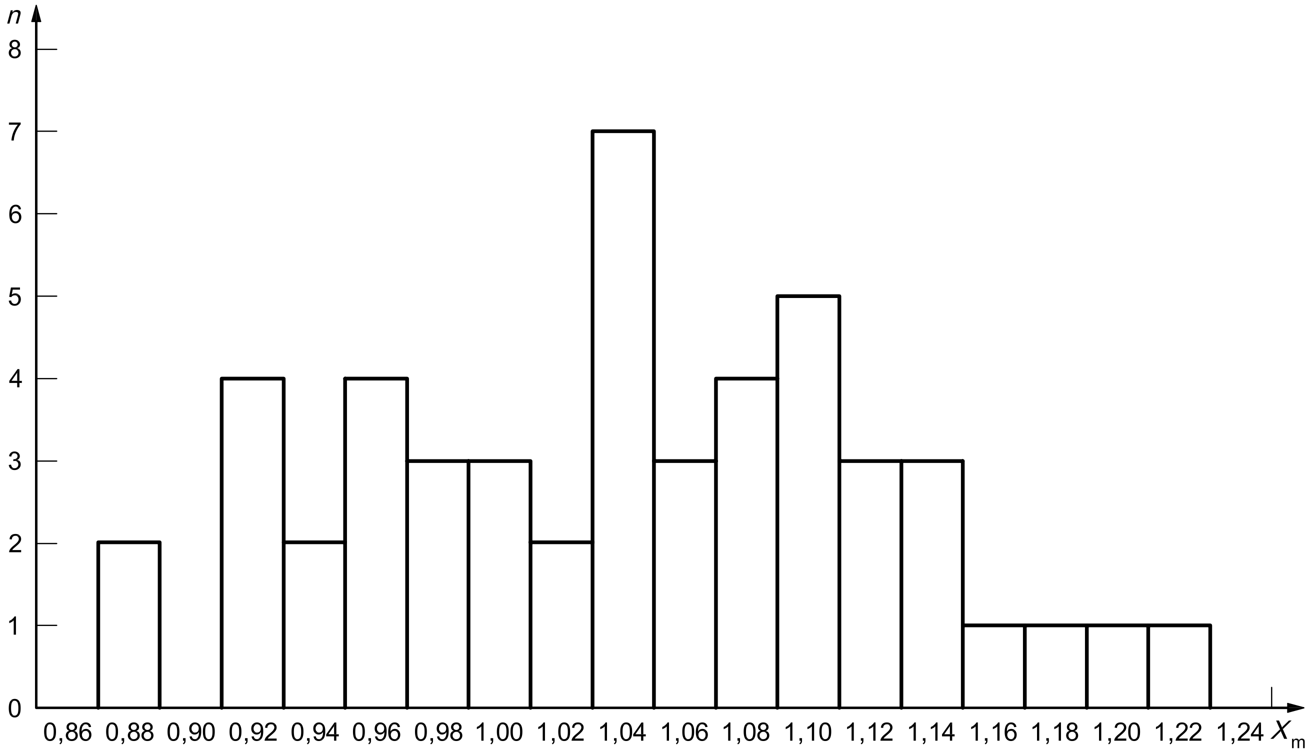
Bias = 1,246 COV = 0,067 $n = 19$

Key

n frequency (number of occurrences)

X_m measured strength/predicted strength

Figure A.13.3-1 — Comparison of measured and predicted strengths of short tubulars subjected to combined compression (local buckling) and bending



Bias = 1,030 COV = 0,082 n = 49

Key

n frequency (number of occurrences)

X_m measured strength/predicted strength

Figure A.13.3-2 — Comparison of measured and predicted strengths of long tubulars subjected to combined compression (column buckling) and bending

A.13.3.4 Piles

No guidance is offered.

A.13.4 Tubular members subjected to combined forces with hydrostatic pressure

A.13.4.1 General

This subclause provides both beam-column stability and strength design interaction equations for the cases in which a tubular member is subjected to axial tension or compression, and/or bending combined with external hydrostatic pressure. The design equations are formulated so that the effects of capped-end actions can be included irrespective of whether they are calculated directly within the structural analysis or not. The purpose of providing equations in this form is to facilitate tubular member design by the two forms of analysis commonly used by designers. In the limit when the hydrostatic pressure is zero, the design equations in this clause reduce to those given in 13.3.

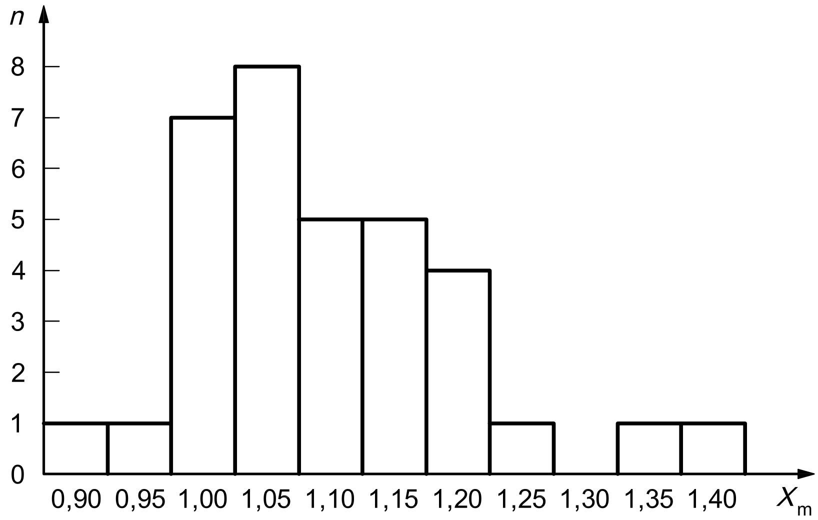
In the equations, hoop compression is not explicitly included in the analysis, but its effect on member design is considered within the design interaction equations. To facilitate the application of these particular equations, the hoop collapse design check stipulated in 13.2.6 should be satisfied first.

A structural analysis in which the capped-end axial compression is explicitly included in the analysis is preferred because it allows for a more precise redistribution of the capped-end forces based on the relative stiffnesses of the braces at a node.

However, as the redistribution of the capped-end axial compression is minimal because of similar brace sizes at a node, the difference between including or excluding the capped-end axial compression in the analysis is usually small. Further discussion of the development of the equations in this subclause can be found in References [A.13.4-1] and [A.13.4-2]. A collection of the test data and additional comparison of the design equations to test data can be found in Reference [A.13.2-1].

A.13.4.2 Axial tension, bending and hydrostatic pressure

Comparisons between test data and predictions using the equations in 13.4.2 are presented in the form of a histogram in Figure A.13.4-1. The statistics of the comparison are also included.



Bias = 1,075 COV = 0,098 n = 34

Key

n frequency (number of occurrences)

X_m measured strength/predicted strength

Figure A.13.4-1 — Comparison of measured and predicted strengths of tubulars subjected to combined tension and external pressure

In the presence of tensile forces in members, external hydrostatic pressure has three main effects:

a) a reduction of the axial tension due to the presence of capped-end axial compression;

b) a reduction of the axial tensile strength (f_t) caused by hoop compression, resulting in $f_{t,h}$;

c) a reduction of the bending strength (f_b) caused by hoop compression, resulting in $f_{b,h}$.

As demonstrated in Reference [A.13.4-2], the axial tension hydrostatic pressure interaction is similar to the bending-hydrostatic pressure interaction. The reduced axial tensile and bending strengths, as given by Equations (13.4-8) and (13.4-9), were derived from the following ultimate strength interaction equations.

Combined axial tension and hydrostatic pressure:

$$\left(\frac{\sigma_t}{f_y}\right)^2 + \left(\frac{\sigma_h}{f_h}\right)^{2\eta} + 2\nu\left(\frac{\sigma_h}{f_h}\right)\left(\frac{\sigma_t}{f_y}\right) = 1{,}0 \qquad \text{(A.13.4-1)}$$

Combined bending and hydrostatic pressure:

$$\left(\frac{\sigma_b}{f_b}\right)^2 + \left(\frac{\sigma_h}{f_h}\right)^{2\eta} + 2\nu\left(\frac{\sigma_h}{f_h}\right)\left(\frac{\sigma_b}{f_b}\right) = 1{,}0 \tag{A.13.4-2}$$

where ν is Poisson's ratio, taken for steel as 0,3 in the derivation of Equations (13.4-8) and (13.4-9).

To obtain the axial tensile and bending strengths from Equations (A.13.4-1) and (A.13.4-2) the σ_t and σ_b terms are represented by $f_{t,h}$ and $f_{b,h}$, respectively, and the positive roots taken of the resulting quadratic equations.

In References [A.13.2-4] and [A.13.4-3], an equation similar to Equation (A.13.4-1), but without the η parameter, was first proposed for the design of tubular members under combined axial tension and hydrostatic pressure. The modified Beltrami-Haigh equation proposed in those references was later found to be too conservative for elastic conditions, such as in the design of tendons of tension leg platforms. So further changes to the modified Beltrami-Haigh equation were made to remove the unnecessary conservatism by introducing the η factor in Equation (A.13.4-1), see Reference [A.13.4-2]. The use of the η factor results in a family of interaction curves representing different hoop collapse to yield strength ratios (f_h/f_y).

When the calculated axial tensile stress is greater than or equal to the capped-end axial compression, i.e. $\sigma_t > \sigma_q$, the member is subjected to net axial tension. For this case, the member is not susceptible to local buckling.

When the calculated axial tensile stress is less than the capped-end axial compression, i.e. $\sigma_t < \sigma_q$, the member is subjected to net axial compression and to a quasi-hydrostatic pressure condition. (A member is subjected to a pure hydrostatic pressure condition when the net axial compressive stress is equal to the capped-end axial stress, i.e. $\sigma_{c,c} = \sigma_q$.) Under this condition there is no member instability. Hence, for the case of $\sigma_t < \sigma_q$ the cross-sectional yield criterion [Equation (13.4-13)] and the cross-sectional elastic buckling criterion [Equation (13.4-18)] should be satisfied.

The test data for tubular members subjected to combined axial tension and hydrostatic pressure only were taken from Reference [A.13.2-20]. A histogram of the comparison between these data and the predictions of Equation (13.4-7) is presented in Figure A.13.4-1. The statistics of the fit are also indicated.

A.13.4.3 Axial compression, bending and hydrostatic pressure

The capped-end axial compression due to hydrostatic pressure does not cause column buckling of a member under combined external compression and hydrostatic pressure. It is therefore incorrect to determine the reduced column buckling strength by subtracting the capped-end axial compression from the in-air buckling strength calculated by Equation (13.2-5). An analogy to the hydrostatic pressure problem is the design of offshore platform conductors, as discussed in Reference [A.13.4-4]. The major contribution of the capped-end axial compression is earlier yielding of the member in the presence of the additional external axial compression. The earlier yielding in turn results in a reduced column buckling strength for the member, as given by Equation (13.4-15). When there is no hydrostatic pressure, i.e. $\sigma_q = 0$, Equation (13.4-15) reduces to the in-air case of Equation (13.2-5).

For the stability check [Equation (13.4-14)], the calculated axial compression (σ_c), which is the external axial compression only, is used. The effect of the capped-end axial compression is captured in Equation (13.4-15) for the representative axial compressive strength in the presence of external hydrostatic pressure ($f_{c,h}$). For the strength or cross-sectional yield check [Equation (13.4-13)], the net axial compression ($\sigma_{c,c}$) of the member is used. In addition, the cross-sectional elastic buckling criterion [Equation (13.4-18)] should be satisfied.

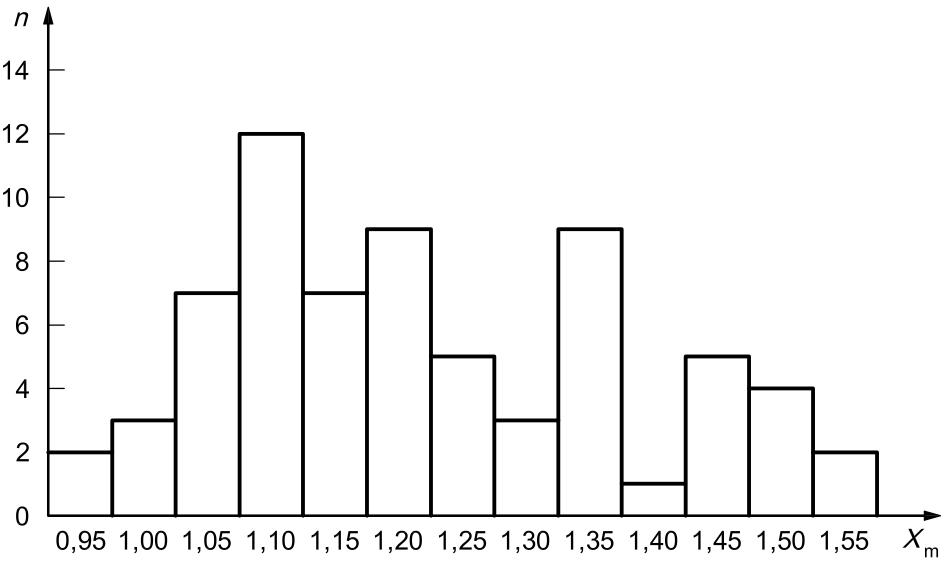
The test data for tubular members subjected to combined axial compression, bending, and hydrostatic pressure were taken from References [A.13.2-7], [A.13.2-13], [A.13.2-14], and [A.13.2-20].

For comparing design equation predictions with test data, a case of combined axial compression, bending, and hydrostatic pressure is used as an example to illustrate the procedure. In a typical test, values of axial compression, bending, external pressure and capped-end stress are measured at member failure. The test values are denoted by $\sigma_{a,test}$ for axial compression, $\sigma_{b,test}$ for bending, $\sigma_{h,test}$ for hoop stress due to external pressure, and $\sigma_{q,test}$ for capped-end stress. All components of the test values contain bias and these biases cannot be accurately determined individually. In the comparison of the test data with the predicted values, it is assumed that the bias for all components is the same (say, X_{bias}).

In the calculation of member utilization ratio using Equations (13.4-19) to (13.4-21), all components of measured test values are modified by dividing them by a constant parameter, X_m, and the modified test values are then substituted into the equations to determine the corresponding utilization ratio. All partial resistance factors should be excluded (i.e. set to 1,0) in the calculation of the utilization ratio. An iterative procedure needs to be used for determining a particular X_m value (X_{bias}) that yields a utilization ratio of 1,0. This value of X_{bias} is a measure of the accuracy of the design equation against the test data in terms of the ratio of test strength to predicted strength. Depending upon the assumed X_m value, some numerical instability can occur, but this should not affect the determination of the correct X_{bias}.

Using Test No. C3JD of the Steinmann and Vojta data [A.13.2-7] as an example, this tubular member has an outside diameter of 168,3 mm, a thickness of 3,96 mm, an overall length of 4 191 mm, Young's modulus of 167 379 N/mm^2, and yield strength of 353,9 N/mm^2. At failure, the measured test components show an axial compressive stress of 149,2 N/mm^2, bending stress of 174,1 N/mm^2, and hydrostatic pressure of 88,7 N/mm^2. Figure A.13.4-4 shows the calculated utilization ratio as a function of the assumed X_m value. The calculated X_m that yields the utilization ratio of 1,0 is 1,25. Note that at failure for this example, the tested hydrostatic pressure slightly exceeds the representative strength.

The comparisons between test data and predictions using the equations are presented in terms of histograms in Figures A.13.4-2 and A.13.4-3 for short and long columns, respectively. The statistics of the comparisons are also included.



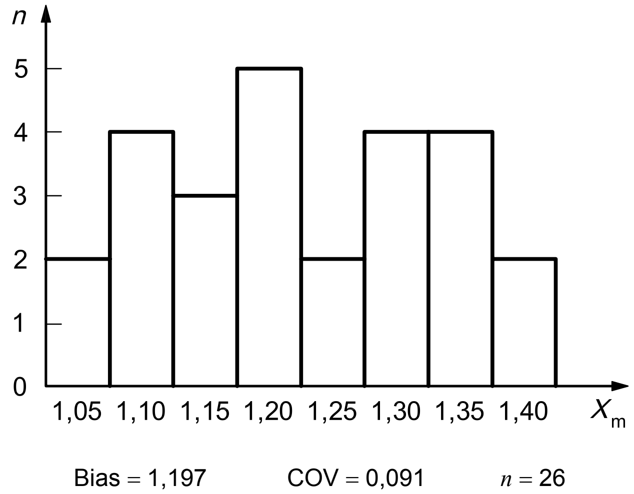
Bias = 1,199 COV = 0,134 n = 69

Key

n frequency (number of occurrences)

X_m measured strength/predicted strength

Figure A.13.4-2 — Comparison of measured and predicted strengths of short columns subjected to combined compression, bending and external pressure

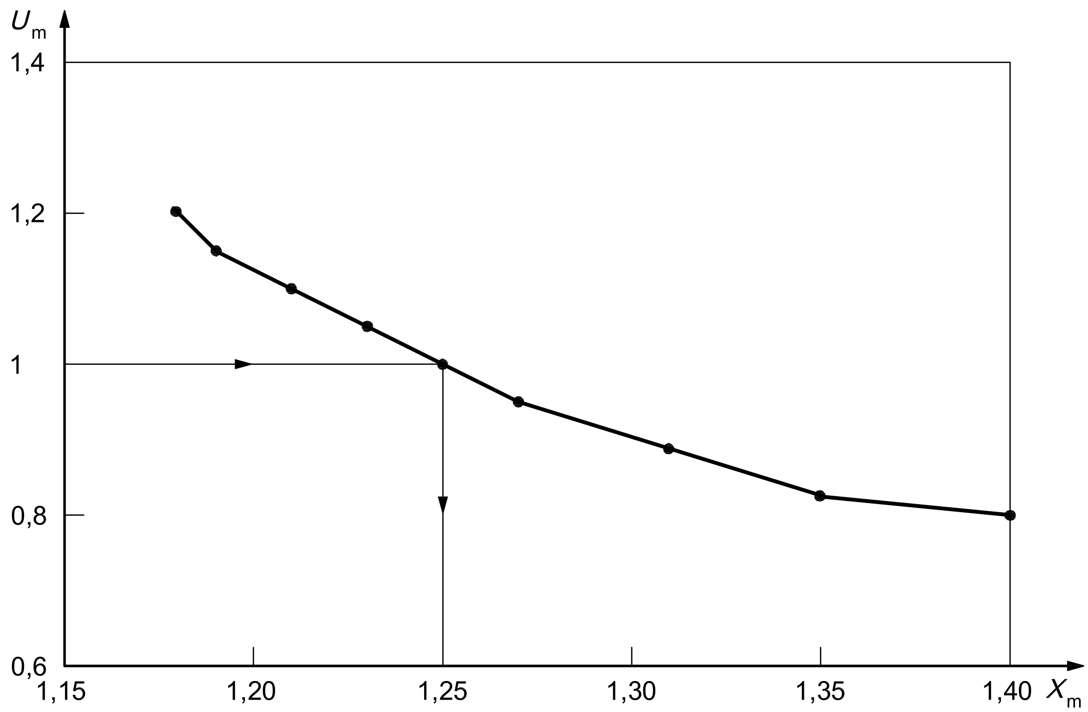


Key

n frequency (number of occurrences)

X_m measured strength/predicted strength

Figure A.13.4-3 — Comparison of measured and predicted strengths of long columns subjected to combined compression, bending and external pressure



Key

U_m utilization ratio

X_m measured strength/predicted strength

Figure A.13.4-4 — Calculated utilization ratio as function of assumed X_m value

A.13.5 Effective lengths and moment reduction factors

The effective length used for a bracing member in framed structures is important for determining its axial compressive strength.

Based on experience with the use of non-linear pushover analyses, it was concluded in Reference [A.13.5-1] and others that the accuracy of linear design predictions is significantly improved by using buckling lengths determined from refined analysis. Studies indicate that buckling lengths for real, three-dimensional structures demonstrate little variation when actions are increased up to collapse. Thus, buckling lengths calculated for the initial elastic loading regime are, in fact, representative throughout the complete loading history.

The effective lengths for initial conditions can be calculated for an X-frame through the use of the differential equations given in Reference [A.13.5-2] or for a general structural component through the use of ϕ functions derived from differential equations, see Reference [A.13.5-3]. Use of ϕ functions implies that exact solutions are obtained for the considered buckling problem. General effective buckling lengths have been derived using ϕ functions incorporating end flexibility of members. The results for an X-brace with four equal-length members are shown in Figure A.13.5-1 as a function of the force distribution in the system, expressed by the ratio Q/P (where P is the maximum compressive force in the X-brace structure), and of the non-dimensional end rotational stiffness parameter, ρ, which is defined by

$$\rho = \frac{CL}{EI} \tag{A.13.5-1}$$

In this equation, C is the rotational stiffness at a node and I is the moment of inertia of the tubular member. C should take into account the local joint flexibility [A.13.5-4] and the flexural and torsional stiffnesses of the incoming members. L refers to centreline-to-centreline distances between end nodes, i.e. excluding the intermediate X-joint node.

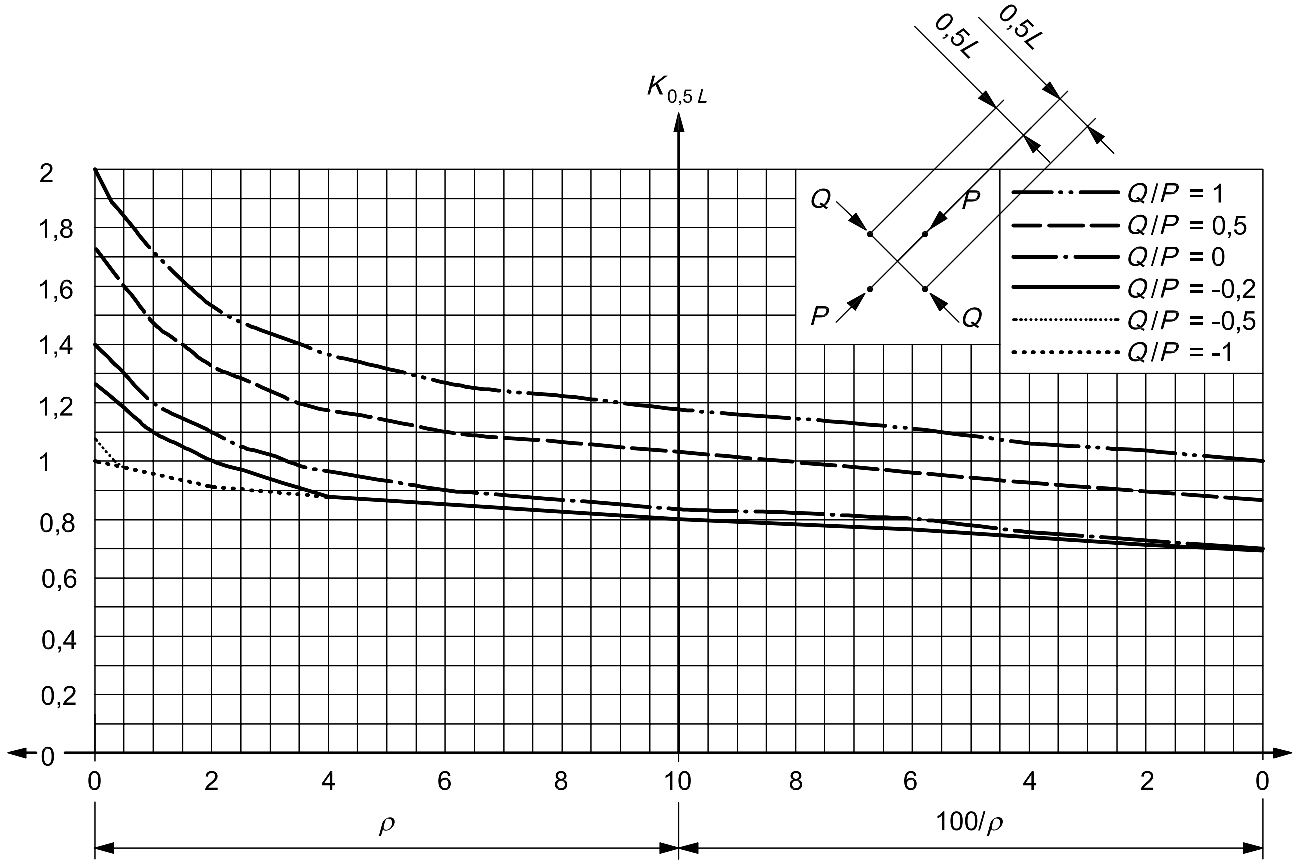
Figures A.13.5-1 to A.13.5-3 provide effective length factors as a function of the rotational stiffness of the member ends and the ratio of the axial forces in the braces for three X-brace configurations:

a) equal segment lengths;

b) a segment length ratio of 0,6 to 0,4;

c) a segment length ratio of 0,7 to 0,3.

In developing Figures A.13.5-1 to A.13.5-3, the local bending stiffness at the X-joint is assumed to be infinite. The torsional stiffness of incoming members at the X-joint has been conservatively neglected.



Key

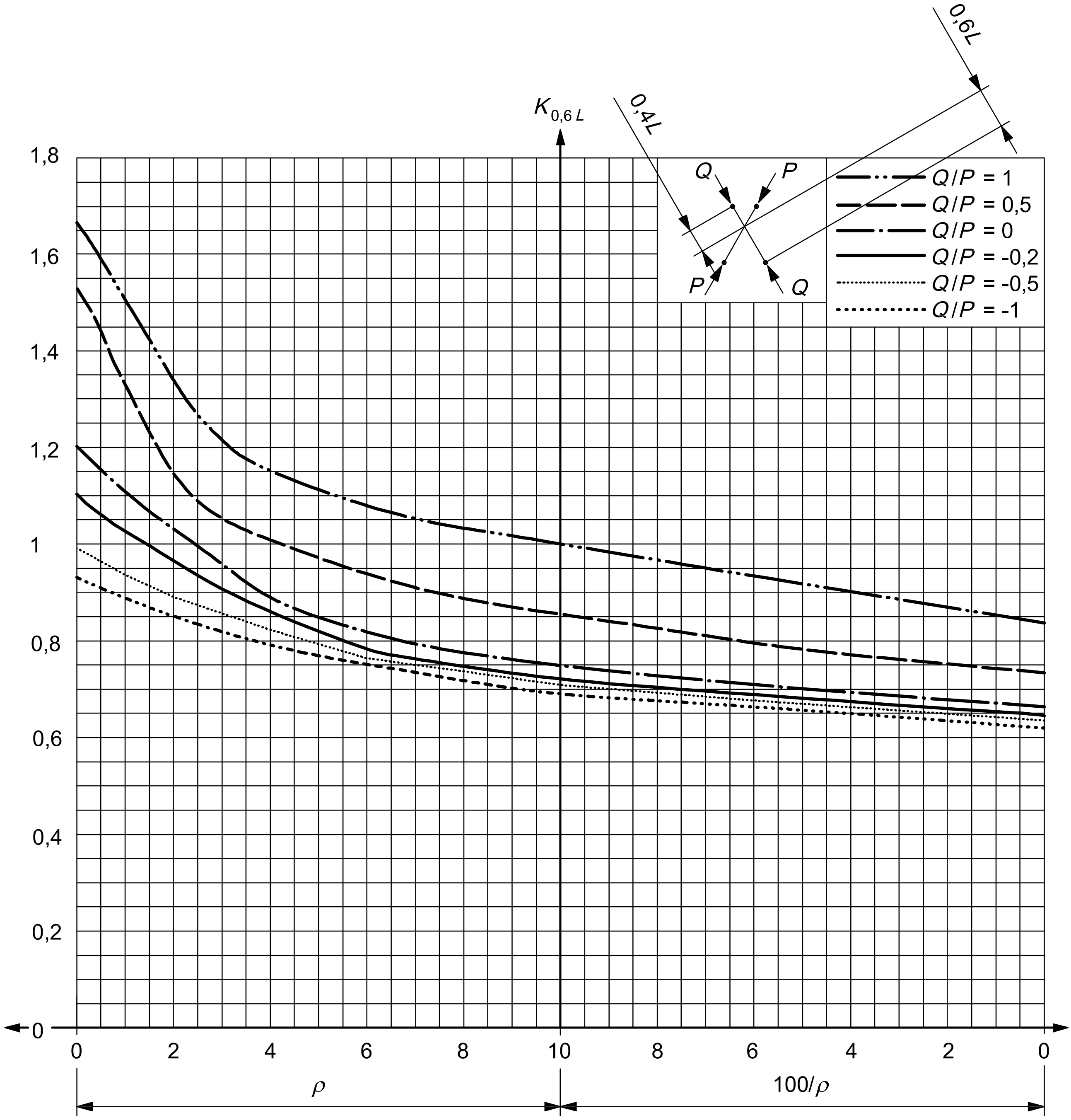
$K_{0,5L}$ effective length factor relative to 50 % of member length

ρ rotational stiffness parameter from Equation (A.13.5-1)

P compressive force in member under consideration

Q compressive force in other member of X frame

Figure A.13.5-1 — Effective length factors for symmetric X-braces as function of force distribution and rotational end stiffnesses for equal lengths (torsional stiffness of crossing member included)



Key

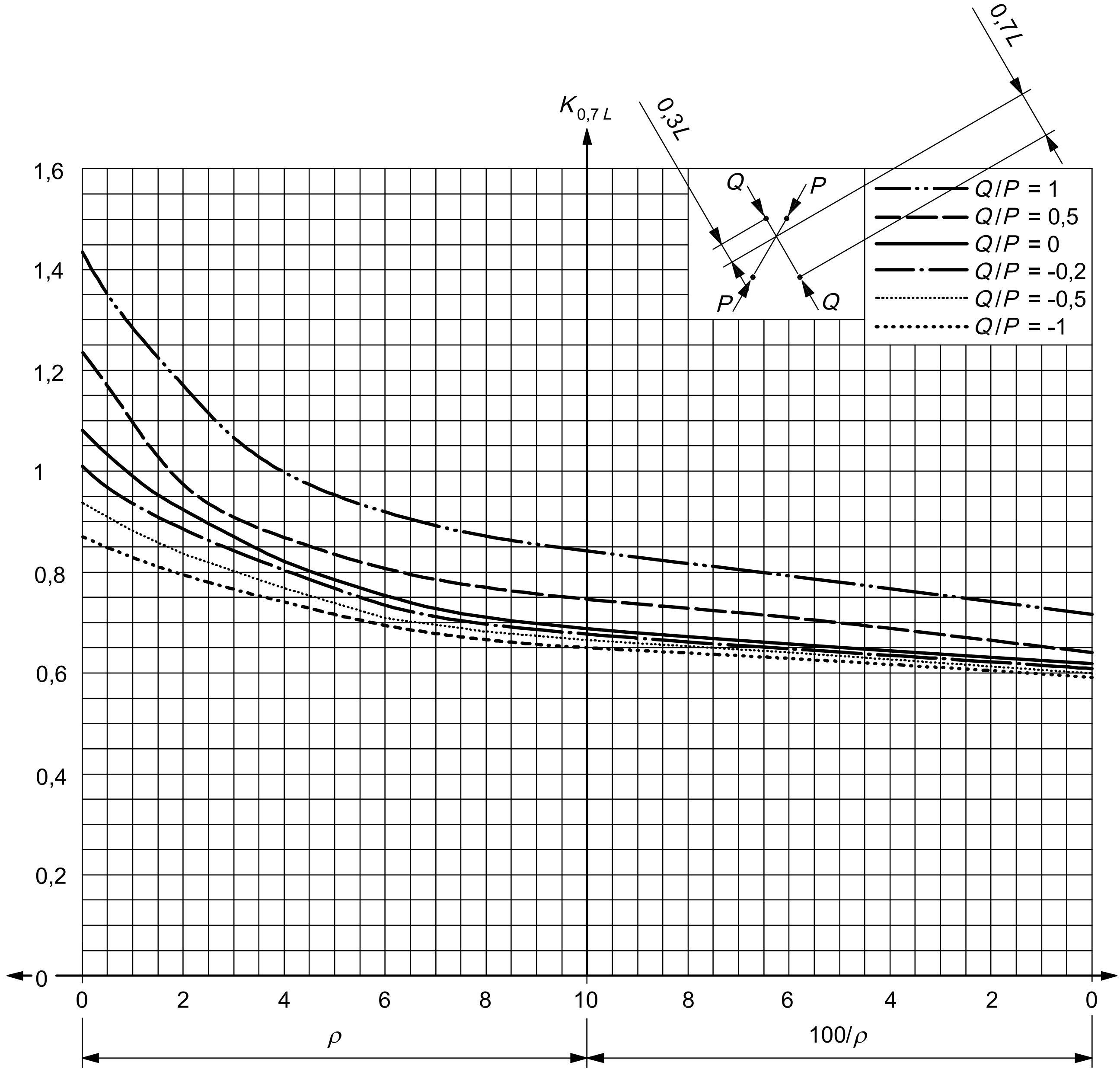
$K_{0,6L}$ effective length factor relative to 60 % of member length

ρ rotational stiffness parameter from Equation (A.13.5-1)

P compressive force in member under consideration

Q compressive force in other member of X frame

Figure A.13.5-2 — Effective length factors for non-symmetric X-braces as function of force distribution and rotational end stiffnesses (longer segment = 0,6 times the brace length)



Key

$K_{0,7L}$ effective length factor relative to 70 % of member length

ρ rotational stiffness parameter from Equation (A.13.5-1)

P compressive force in member under consideration

Q compressive force in other member of X frame

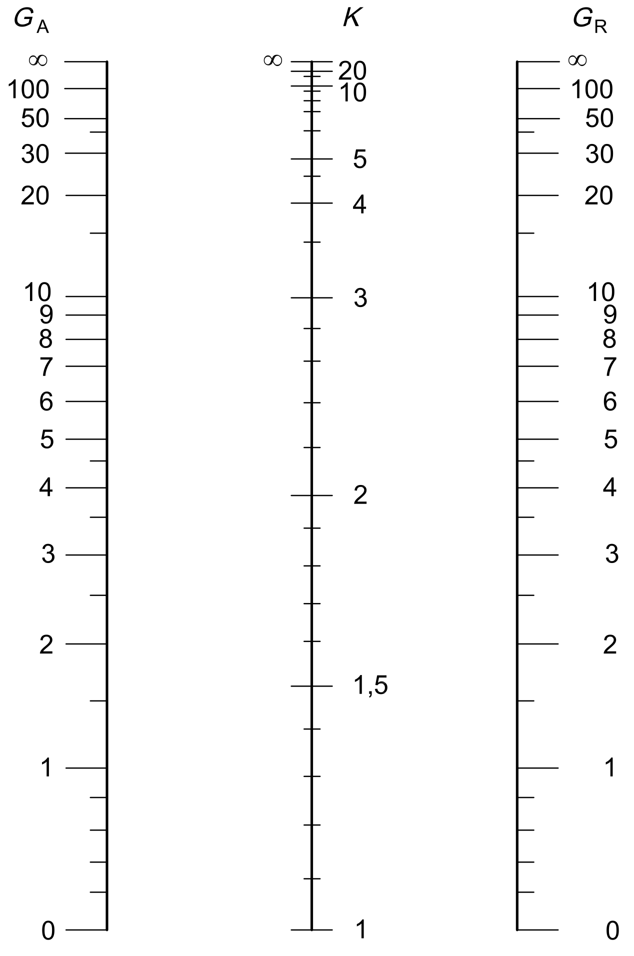
Figure A.13.5-3 — Effective length factors for non-symmetric X-braces as function of force distribution and rotational end stiffnesses (longer segment = 0,7 times the brace length)

Figure A.13.5-1 indicates that for realistic end conditions ($\rho > 3$) and for single braces, i.e. when $Q/P = 1$ (if $Q/P = 1,0$, the member with Q load has no effect on the P loaded member), a K factor with respect to the full X-brace length of less than 0,8 (i.e. less than 1,6 of 0,5 L) is acceptable provided end joint stiffnesses do not diminish as the braces become fully loaded. Experimental results relating to frames seem to confirm 0,7 with respect to the full brace length is acceptable, which is confirmed by the results in Figure A.13.5-1 and recommended in Table 13.5-1.

In Table 13.5-1, a K factor of 0,8 with respect to the longer segment length is recommended for X-braces in which $Q/P \leqslant 0$. This value is well supported by the results in Figure A.13.5-3 for unequal X-braces with realistic end conditions ($\rho = 3$ to 5). For equal X-braces, however, an unusually high rotational end stiffness is required ($100/\rho < 7$) to obtain a K factor of 0,8 in Figure A.13.5-1 — notwithstanding that the experimental results studied in Reference [A.13.5-5] support a K factor value of 0,8 for equal X-braces too. This may be attributed to the fact that significant rotational restraint is provided at the X-joint and at member ends.

For cases in which both X-braces are in compression, for realistic end conditions a K factor of 0,7 with respect to the full member length may be used. Implicitly, the X-joint is not braced.

To estimate the effective length of an unbraced column, such as topsides legs, the use of the alignment chart in Figure A.13.5-4 provides a fairly rapid method for determining adequate K values. The alignment chart may be modified to allow for conditions different from those assumed in developing the chart.



The subscripts A and R refer to the joints at the two ends of the column section being considered. G is defined as

$$G = \frac{\dfrac{I_c}{L_c}}{\Sigma \dfrac{I_0}{L_0}}$$

where Σ indicates a summation of all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered, I_c is the moment of inertia, L_c the unsupported length of the column section, I_0 is the moment of inertia and L_0 the unsupported length of a girder or other restraining member. I_c and I_0 are taken about axes perpendicular to the plane of buckling being considered.

For column ends supported by, but not rigidly connected to, a footing or foundation, G is theoretically infinity; but, unless actually designed as a true friction free pin, it may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1,0. Smaller values may be used if justified by analysis.

Figure A.13.5-4 — Alignment chart for determining the effective length of unbraced columns

For cantilever tubular members, an independent rational analysis is required for the determination of appropriate effective length factors. Such an analysis should take full account of all large deflection (P-Δ) effects. For a cantilever tubular member, $C_m = 1{,}0$, where C_m is the moment reduction factor.

The purpose of using of C_m in the interaction Equations (13.3-3) and (13.4-14) is to obtain an equivalent moment for the moment pattern to which the beam–column is subjected that more accurately reflects the effect of the moment pattern on beam-column buckling of the member. Further discussion can be found in Reference [A.13.5-6], where more accurate expressions are provided — particularly for relatively low levels of axial compression. The C_m values recommended in Table 13.5-1 are similar to those recommended in Reference [A.13.5-7] and its commentary.

A.13.6 Conical transitions

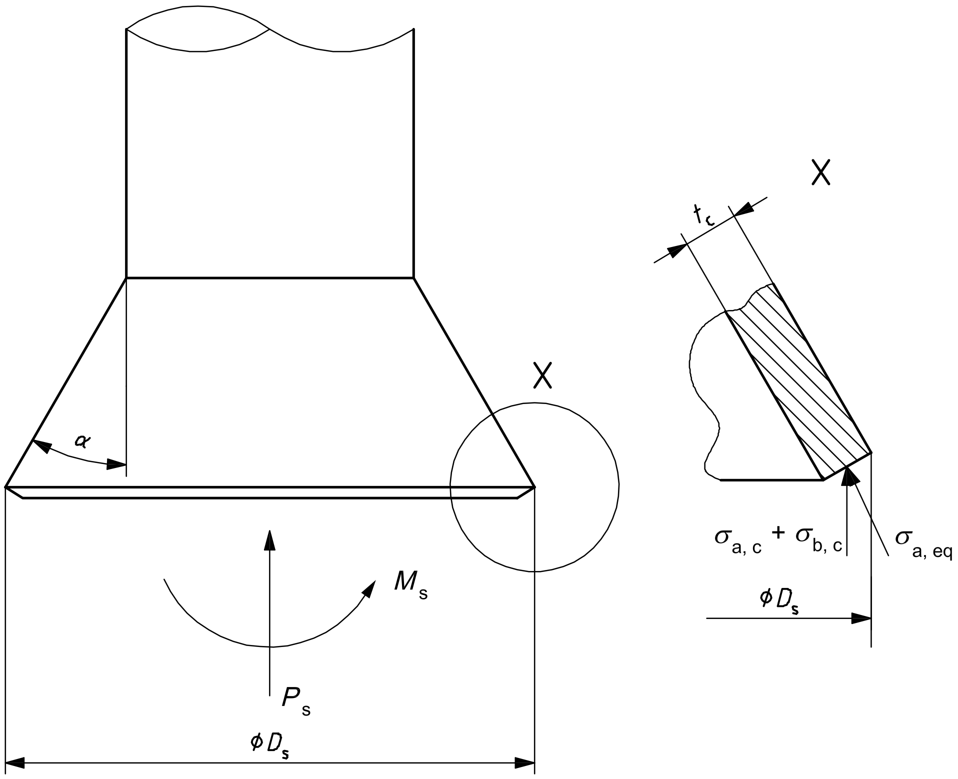
A.13.6.1 General

No guidance is offered.

A.13.6.2 Design stresses

A.13.6.2.1 Equivalent axial stress in conical transitions

The relationship between global section forces and axial stresses in a conical transition is shown in Figure A.13.6-1. The axial stress due to global bending is considered to be of similar nature to that of the compressive stress, although at the opposite side of the conical transition it is tensile.



Key

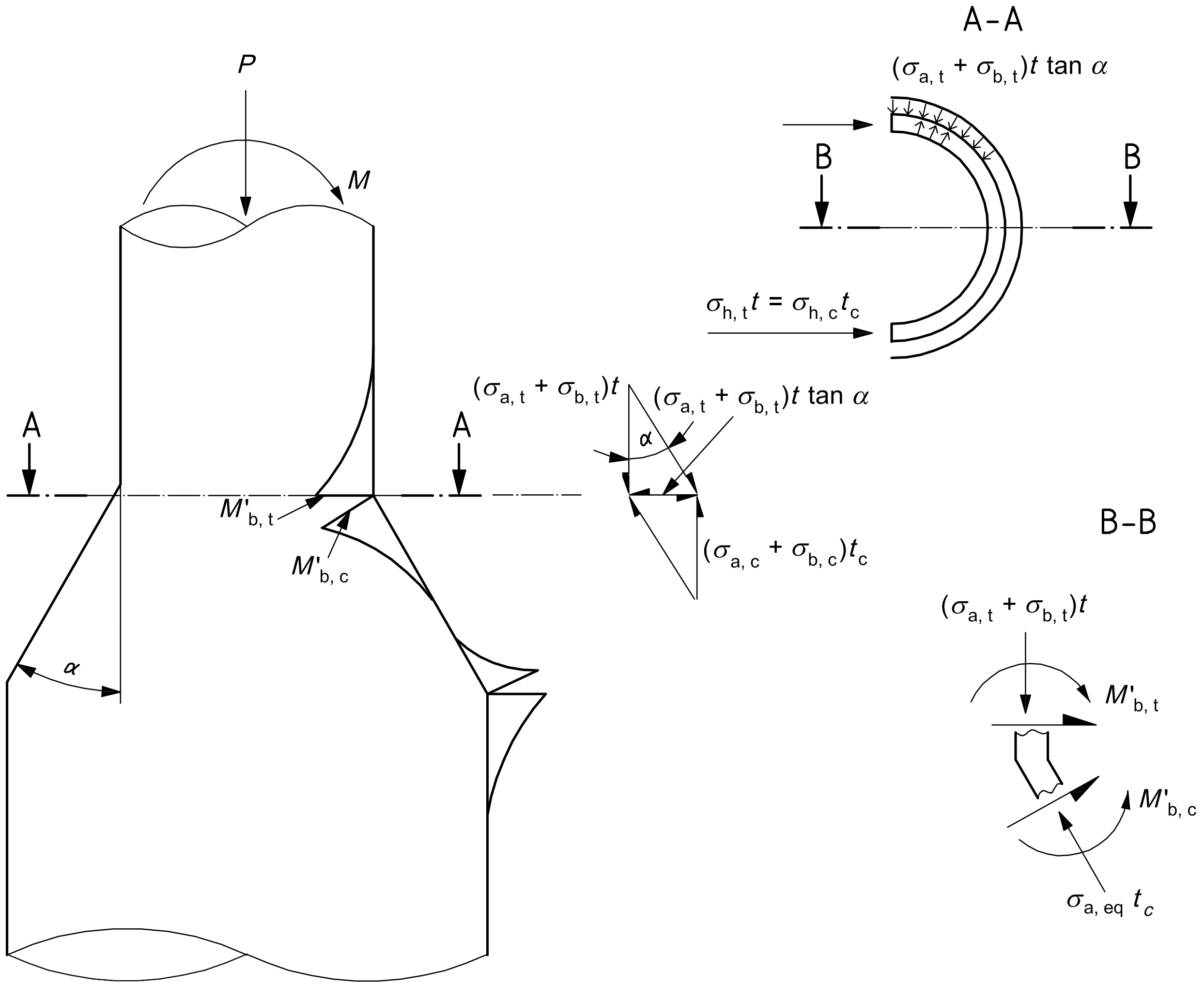
M_s bending moment on section under consideration due to factored actions
P_s axial force on section under consideration due to factored actions
D_s outer diameter of cone section under consideration
α slope angle of conical section
t_c thickness of cone at section under consideration
$\sigma_{a,c}$ axial stress at conical section under consideration
$\sigma_{b,c}$ bending stress at conical section under consideration
$\sigma_{a,eq}$ equivalent axial stress at conical section under consideration

Figure A.13.6-1 — Relationship between section forces and axial stresses in a conical transition

A.13.6.2.2 Local stresses at unstiffened junctions

The radial force component per unit circumferential length at a junction (see the right hand side of Figure A.13.6-2) is proportional to the global longitudinal stress in the tubular due to factored actions and the tangent of the cone angle, i.e. radial force $\propto (\sigma_{a,t} + \sigma_{b,t})t \tan\alpha$, see Figure A.13.6-2. This radial force is additional to any action effects caused by hydrostatic pressure.

The radial force results in a local bending moment and associated shear forces at the junction. The resulting local bending stresses in the tubular and the cone sides of the junction are given by Equations (13.6-4) and (13.6-5). Both these equations are based on results presented in Reference [A.13.6-1].



Key

M bending moment on member about cone/tubular junction due to factored actions

P axial force on member due to factored actions

t thickness of tubular at junction

t_c thickness of cone at junction

α slope angle of conical section

$M'_{b,t}$ bending moment in tubular at junction for unit circumference

$M'_{b,c}$ bending moment in cone at junction for unit circumference

$\sigma_{a,t}$ axial stress in tubular section at junction

$\sigma_{b,t}$ bending stress in tubular section at junction

$\sigma_{a,c}$ axial stress in cone section at junction

$\sigma_{b,c}$ bending stress in cone section at junction

$\sigma_{a,eq}$ equivalent axial stress

NOTE The bending moment distributions presented here are approximate.

Figure A.13.6-2 — Local bending stress and hoop stress at unstiffened junction

A.13.6.3 Strength requirements without external hydrostatic pressure

A.13.6.3.1 General

No guidance is offered.

A.13.6.3.2 Local buckling within conical transition

For local buckling under axial compression and bending, conical transitions with an apex angle less than 60° may be considered as equivalent cylinders with the local cone wall thickness and a diameter equal to $D_s/\cos\alpha$. For conical transitions of constant wall thickness, it is conservative to use the diameter at the larger end of the cone for D_s.

Offshore structures generally have a very small number of cones. Thus, it can be expeditious to design the cones with a geometry such that the representative local buckling strength of the conical transition is equal to the yield strength, i.e. use Equation (13.2-8) given in 13.2.3.3.

A.13.6.3.3 Junction yielding

When the hoop stress is tensile, the strength of the junction is determined by a yield check in accordance with the von Mises-Hencky yield criterion.

A.13.6.3.4 Junction buckling

The strength of the junction with regard to buckling is checked for the absolute largest value of the compressive hoop stress. Equation (13.6-15) is the same as Equation (A.13.4-1), apart from the addition of partial resistance factors in the former. The value suggested for the calculation of f_{he} is based on the results in Reference [A.13.6-1].

A.13.6.3.5 Junction fatigue

The stress concentration factors are derived directly as the ratio between the maximum global plus local stresses normal to the junction and the global axial stresses in the tubular or the cone, respectively, as derived from forces due to factored actions.

A.13.6.4 Strength requirement with external hydrostatic pressure

A.13.6.4.1 Hoop buckling

Hoop buckling is analysed in a similar manner to that for a tubular subjected to external hydrostatic pressure, using equivalent geometry properties.

A.13.6.4.2 Junction yielding and buckling

The action effect from external hydrostatic pressure is added directly to the other stresses at the junction when performing the utilization checks with respect to yielding and buckling.

A.13.6.5 Ring design

No guidance is offered.

A.13.7 Dented tubular members

A.13.7.1 General

The design recommendations are tailored to dented tubulars with dimensions and material yield strengths typical of offshore structure members (f_y < 500 MPa and $D/t \leqslant 120$). Because of the lack of test data for larger dent depths, the applicability of the proposed equations for dented members should be limited to $h/D \leqslant 0{,}3$ and $h/t \leqslant 10$.

The most significant parameter affecting the residual strength of dented members is dent depth. Experimental and analytical data — see References [A.13.7-1] to [A.13.7-4] — have shown that dent length and shape all have relatively negligible effects on the strength of a dented member. Furthermore, provided the dent is located length-wise in the middle half of the member, the loss of strength in a dented member seems insensitive to dent location, Reference [A.13.7-2].

The equations in this subclause are based largely on Reference [A.13.7-5], although the force format adapted therein has been converted to a "stress" format for inclusion in this International Standard.

A.13.7.2 Dented tubular members subjected to tension, compression, bending or shear

A.13.7.2.1 General

The tension, compression and bending recommendations contained in this subclause are based on a programme of testing carried out since the late 1970s. Hydrostatic pressure recommendations have not been included in 13.7 due to limited data. Applicable tests up to 1995 have been documented in the references cited. Figures showing test results supporting the recommendations of subclause 13.7 are included below in the relevant subclauses.

A.13.7.2.2 Axial tension

The strength equation for axial tension relies on yielding of the steel of the dented cross-section whose cross-sectional area is assumed unaltered by the damage.

A.13.7.2.3 Axial compression

Dented tubular members subjected to axial compression fail primarily due to overall column buckling, with or without local buckling. The capacity of the dented member has been reduced by ξ_c with respect to the undamaged cross-section to account for the reduced effective cross-sectional area of the dented section.

The parameter ξ_c, defined in Equation (13.7-14), is discussed below.

a) Column buckling

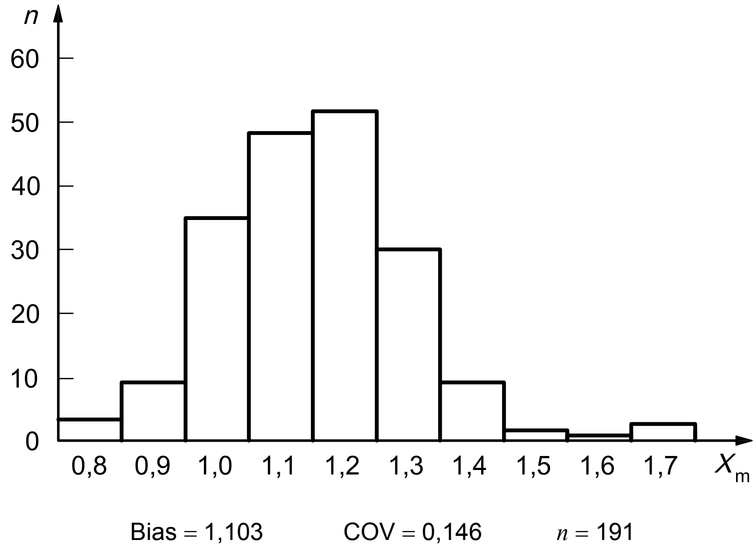
The representative axial compressive strength for members with an out-of-straightness, Δy, which is not necessarily a consequence of the damage, is determined from Equations (13.7-5) and (13.7-6), which account for the additional bending stress on the dented cross-section due to the member out-of-straightness. Equation (13.7-5) deals with an out-of-straightness for dented tubulars that is still within the corresponding fabrication tolerance for undamaged members, i.e. 0,001 of the length. As such, only the effect of the reduction in effective cross-sectional properties at the dent needs to be taken into account. For an out-of-straightness greater than the fabrication tolerance, only the out-of-straightness in excess of the tolerance needs to be considered [Equation (13.7-6)] because the column buckling Equations (13.2-5) and (13.2-6) already account for Δy values up to the level of the tolerance.

The column buckling equations are similar to the undamaged member equations [Equations (13.2-5) and (13.2-6)] except that λ has been replaced by λ_d, the column slenderness parameter that accounts for the effects of denting on cross-sectional properties. The test database was obtained from several different investigators, see References [A.13.2-6] , [A.13.2-9], [A.13.7-1] to [A.13.7-4] and [A.13.7-6] to [A.13.7-13]. A comparison between the test data and the predictions using Equations (13.7-5) and (13.7-6) is given in Figure A.13.7-1.

b) Local buckling

The local buckling equation for dented members is similar to that for undamaged tubular members except that the parameter ξ_c has been introduced [Equations (13.7-7) and (13.7-8)] to account for the reduced strength of the dented cross-section. Discussion of the dented member parameters ξ_c and ξ_m (see A.13.7.2.4) can be found in Reference [A.13.7-14].

The results of column tests on relatively short damaged tubulars ($\lambda_d < 0,252$) are plotted normalized with respect to their undamaged strength [Equations (13.2-8) and (13.2-9)] against the dent damage ratio h/t in Figure A.13.7-2. Also plotted is the cross-sectional area dented member parameter, ξ_c, [Equation (13.7-14)]. The appropriateness of ξ_c to represent the effects of dent damage on the local buckling strength is clearly seen.

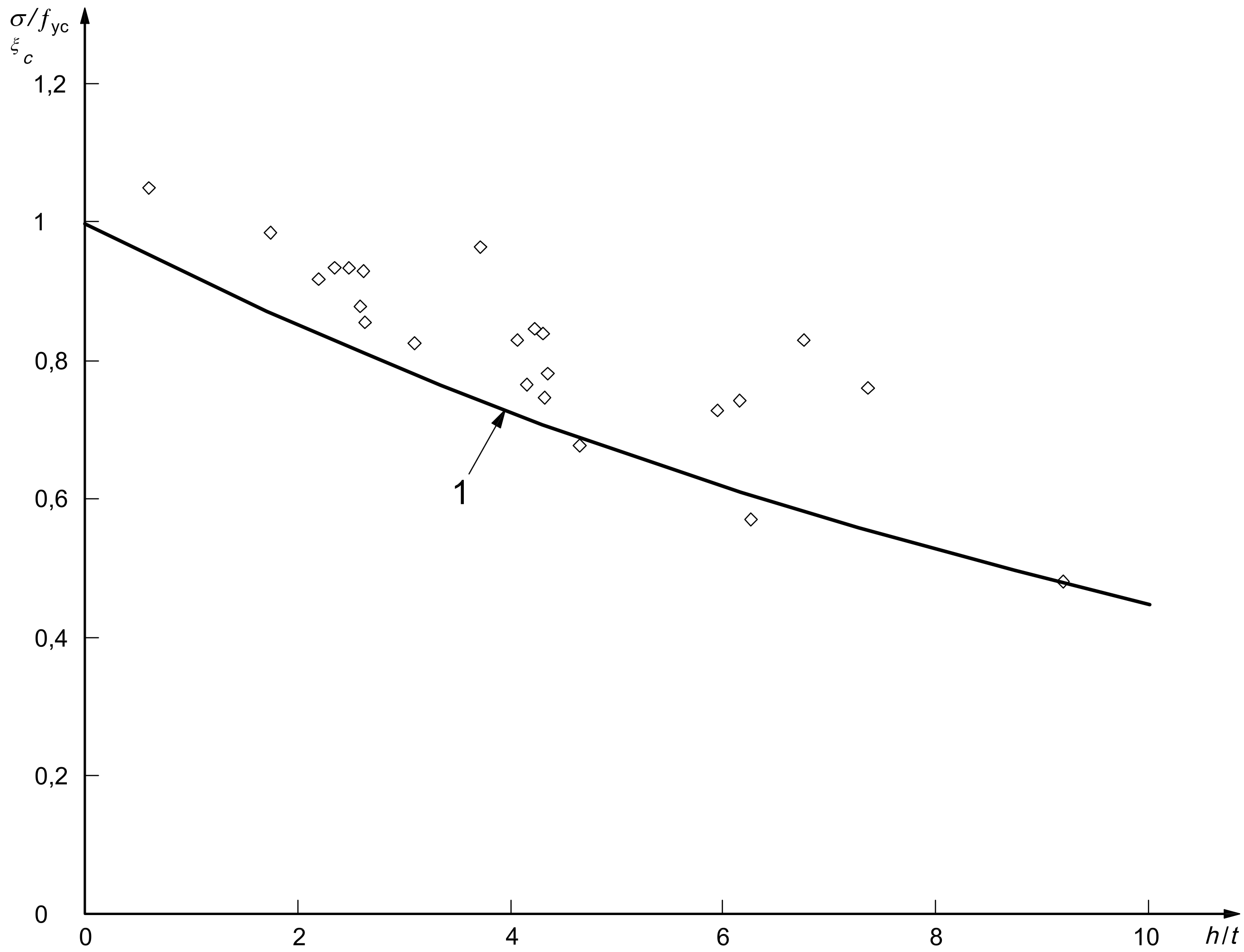


Key

n frequency (number of occurrences)

X_m measured strength/predicted strength

Figure A.13.7-1 — Comparison of measured and predicted strengths for dented tubulars subjected to axial compression



Key

1 $e^{-0,08\,h/t}$

h dent depth

t wall thickness

σ stress in test member

f_{yc} representative local buckling strength (undamaged)

ξ_c compression strength knock-down factor

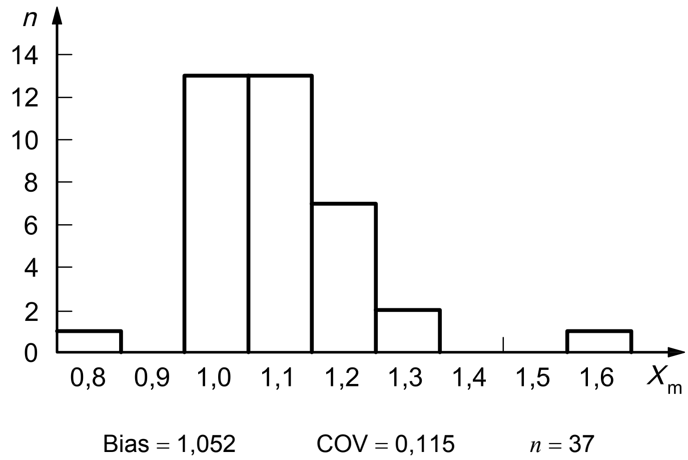
Figure A.13.7-2 — Comparison of normalized test data of dented tubulars with compression strength knock-down factor for $\lambda_d < 0,252$

A.13.7.2.4 Bending

When dented tubulars are subjected to bending moments resulting in tensile (positive) or zero (neutral) stresses in the centre region of the dent, the bending strength of the dented tubular is not appreciably lower than that of an undamaged member. The representative bending strength equations for members under positive [Equation (13.7-16) or neutral (Equation (13.7-20)] bending are thus similar to that for undamaged members [Equation (13.2-11)].

For members subjected to bending moments resulting in compressive (negative) stress in the centre region of the dent, the bending strength of the dented member is reduced compared with that of the undamaged member. Test data from References [A.13.7-9] and [A.13.7-15] indicate that the reduction in strength can be approximated by the application of Equation (13.7-15) to Equation (13.2-11).

A comparison between test data and the prediction of the representative bending strength equation, $\xi_m \times f_b$, is plotted in Figure A.13.7-3. The statistics of the fit are indicated on the figure.



Key

n frequency (number of occurrences)

X_m measured strength/predicted strength

Figure A.13.7-3 — Comparison of measured and predicted strengths for dented tubulars subjected to bending (dent in compression)

A.13.7.2.5 Shear

No guidance is offered.

A.13.7.3 Dented tubular members subjected to combined forces

A.13.7.3.1 Axial tension and bending

The tension-bending interaction equations for dented tubulars, Equations (13.7-24) and (13.7-26), are similar in form to the corresponding equation for undamaged members, Equation (13.3-1); although, for Equation (13.7-26), a less compact format is used because of the need to cater to the differences between positive and negative bending effects. The α term in Equation (13.7-26) reflects the change in the interaction between the two orthogonal moments when a dent is present, see Reference [A.13.7-14].

Recommendations for the interaction of axial tension, bending, and hydrostatic loads have not been included due to limited test data.

A.13.7.3.2 Axial compression and bending

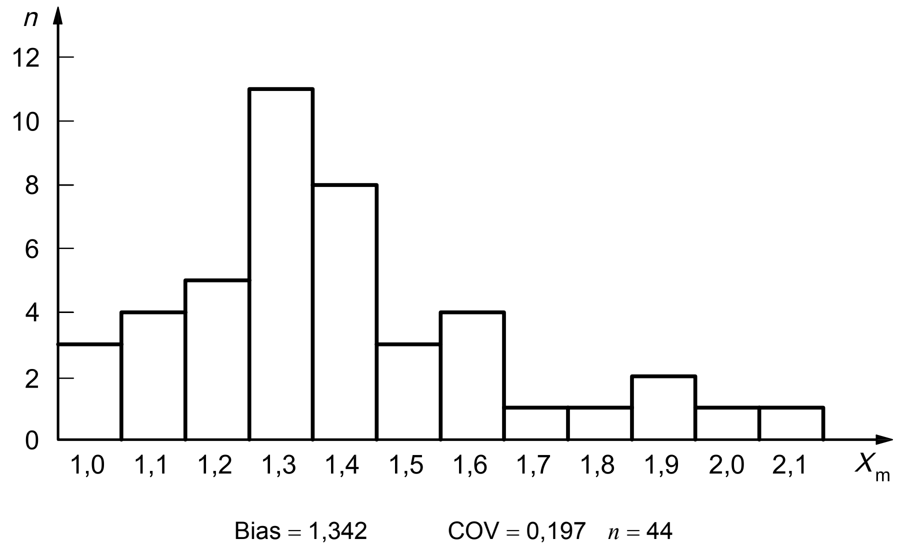
Both overall beam-column stability checks, Equations (13.7-29) and (13.7-34), and strength checks, Equations (13.7-30) and (13.7-35), for dented tubular members subjected to combined axial compression and bending are provided, again in a format similar to that accepted for undamaged tubulars in 13.3.3.

For members with different effective length factors in the negative and neutral bending directions, the Euler buckling strengths, f_e and $f_{e,d}$, will be different and should be taken into account in Equation (13.7-34).

The value of C_m used in the stability equations for dented tubulars has been conservatively set to 1,0, although an upper limit of 0,85 is normally recommended for end-restrained undamaged members in laterally braced frames typical of offshore steel structures as described in Table 13.5-1. For dented members, values of C_m smaller than 1,0 may be justified, but these should be considered on a case-by-case basis.

A comparison between test data and the combined axial compression and bending strength equations, Equations (13.7-29), (13.7-30), (13.7-34) and (13.7-35), is plotted in Figure A.13.7-4. The statistics of the fit are indicated in the figure.

Recommendations for the interaction of axial compression, bending, and hydrostatic loads have not been included due to limited test data.



Key

n frequency (number of occurrences)

X_m measured strength/predicted strength

Figure A.13.7-4 — Comparison of measured and predicted strengths for dented tubulars subjected to combined compression and bending (dent in tension, compression or on neutral axis)

A.13.8 Corroded tubular members

One approach to estimate the strength of an approximately uniformly corroded member is to assume a reduced thickness for the entire member. The reduced thickness should be consistent with the average material loss due to corrosion. The member with the reduced thickness can then be evaluated as an undamaged member. This reduced thickness approach is generally conservative, as discussed in Reference [A.13.8-1]. However, caution should be exercised with respect to fatigue.

Another common case of corroded members is the presence of severe localized corrosion in the form of patches. This form of corrosion cannot be approximated as uniform. References [A.13.7-13], [A.13.8-2] and [A.13.8-3] provide equations to estimate the local buckling strength of these corroded members as a function of the thickness loss. No information is available on the effect of localized corrosion on overall member strength. In this case, finite element analysis or a numerical segment approach based on empirical or analytical moment-axial force-curvature ($M - P - \Phi$) relations can be used. For fatigue sensitive conditions, a fatigue evaluation of the corroded member should also be considered.

A.13.9 Grouted tubular members

A.13.9.1 General

Grout filling offers benefits to undamaged members, but offers greater benefits to members suffering limited damage following impacts from vessels or dropped objects. For undamaged members, greater member strength and stiffness can be achieved at the expense of weight, but without any corresponding increase in environmental actions. For damaged members, growth in dent damage is inhibited, enabling the full strength of the damaged cross-section to be achieved.

Most of the available test data relates to tubulars in which grouting has been introduced following damage, usually in the form of local dents but frequently accompanied by overall bowing. The reason for this is that the use of grouting in offshore tubulars has largely been driven by the need to strengthen/stiffen damaged members.

The formulations presented in 13.9 are only applicable to fully grouted tubular members. Based on actual experience, it can be difficult to achieve a fully grouted condition in offshore repair or strengthening operations. Consequently, some reduction in the calculated residual strength will be necessary to account for the potential incomplete grouting at the end of the grouted member. Because only limited data are available for partially grouted tubular members (especially undamaged ones) and because defining the extent of the grout filling of tested specimens is not precise, no design equations for partially grouted tubular members can be given at this time. However, the strength equations for such partially filled tubulars can be formulated by assuming that the grout does not contribute to either the axial compressive strength or to the moment capacity. This is achieved by setting the terms for the fully grouted members that relate specifically to the grout to zero, see References [A.13.9-1] and [A.13.9-2].

A.13.9.2 Grouted tubular members subjected to tension, compression or bending

A.13.9.2.1 General

The tension, compression and bending requirements contained in this subclause are based largely on the studies reported in References [A.13.9-1] and [A.13.9-2]. Shear and hydrostatic pressure requirements are not included because of lack of data. Applicable tests up to 1995 have been documented in the above references. Figures showing test results supporting the requirements of 13.9 are included in the appropriate subclauses.

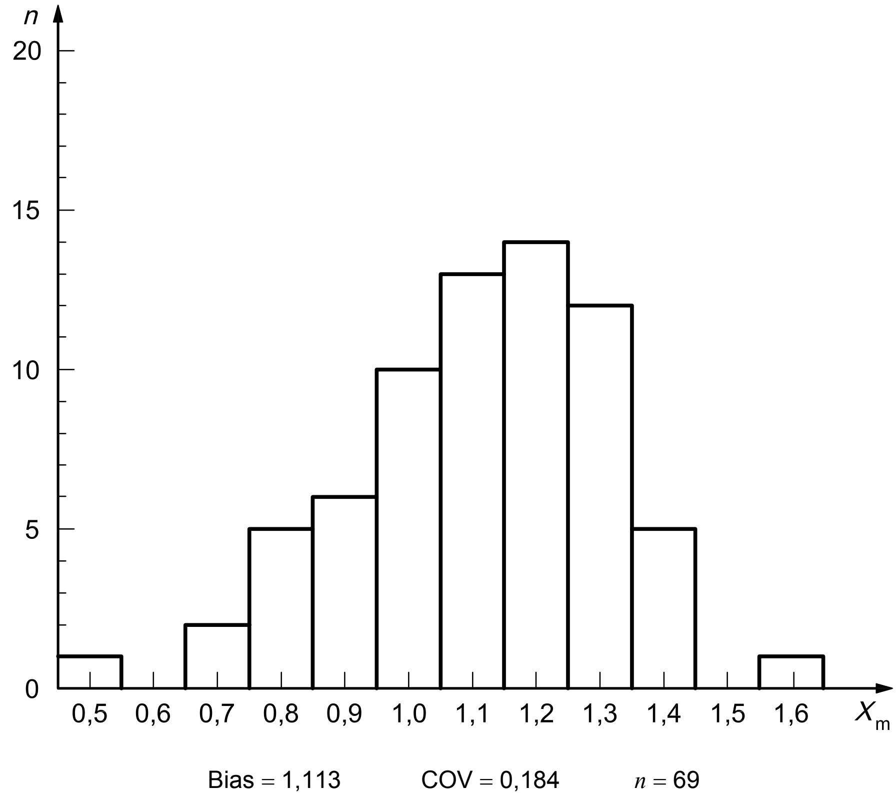
A.13.9.2.2 Axial tension

The strength equation for axial tension reflects only yielding of the steel cross-section. The contribution of the grout to the tensile capacity of the fully grouted member has been conservatively ignored. When complete grouting can be demonstrated, advantage can be taken of the benefits of the grouting.

A.13.9.2.3 Axial compression

The axial compressive strength of fully grouted tubular members is determined from Equations (13.9-8) and (13.9-9). These column buckling equations are similar to the undamaged and non-grouted member strength equations [Equations (13.2-5) and (13.2-6)], except that the slenderness parameter λ has been replaced by λ_g and the representative local buckling strength f_{yc} has been replaced by the axial squash strength f_{ug} of the fully grouted member.

The test data for fully grouted undamaged tubulars are taken from References [A.13.9-1] and [A.13.9-3] to [A.13.9-5]. Similar data for fully grouted damaged tubulars are taken from References [A.13.9-3] to [A.13.9-8]. A comparison between the test data and the predictions using the strength equations is presented in Figure A.13.9-1.



Key

n frequency (number of occurrences)

X_m measured strength/predicted strength

Figure A.13.9-1 — Comparison of measured and predicted strengths using the representative axial compressive strength equations for fully grouted damaged and undamaged tubular members

A.13.9.2.4 Bending

The bending strength of fully grouted tubular members may be calculated from Equation (13.9-19). The bending strength of the dented section is found from a second order fit to the exact equation, simply expressed in terms of dent depth to diameter ratio. The effect of the presence of the grout is based on an approximation to the semi-graphical representation of this feature in BS 5400 [A.13.9-9].

A.13.9.3 Grouted tubular members subjected to combined forces

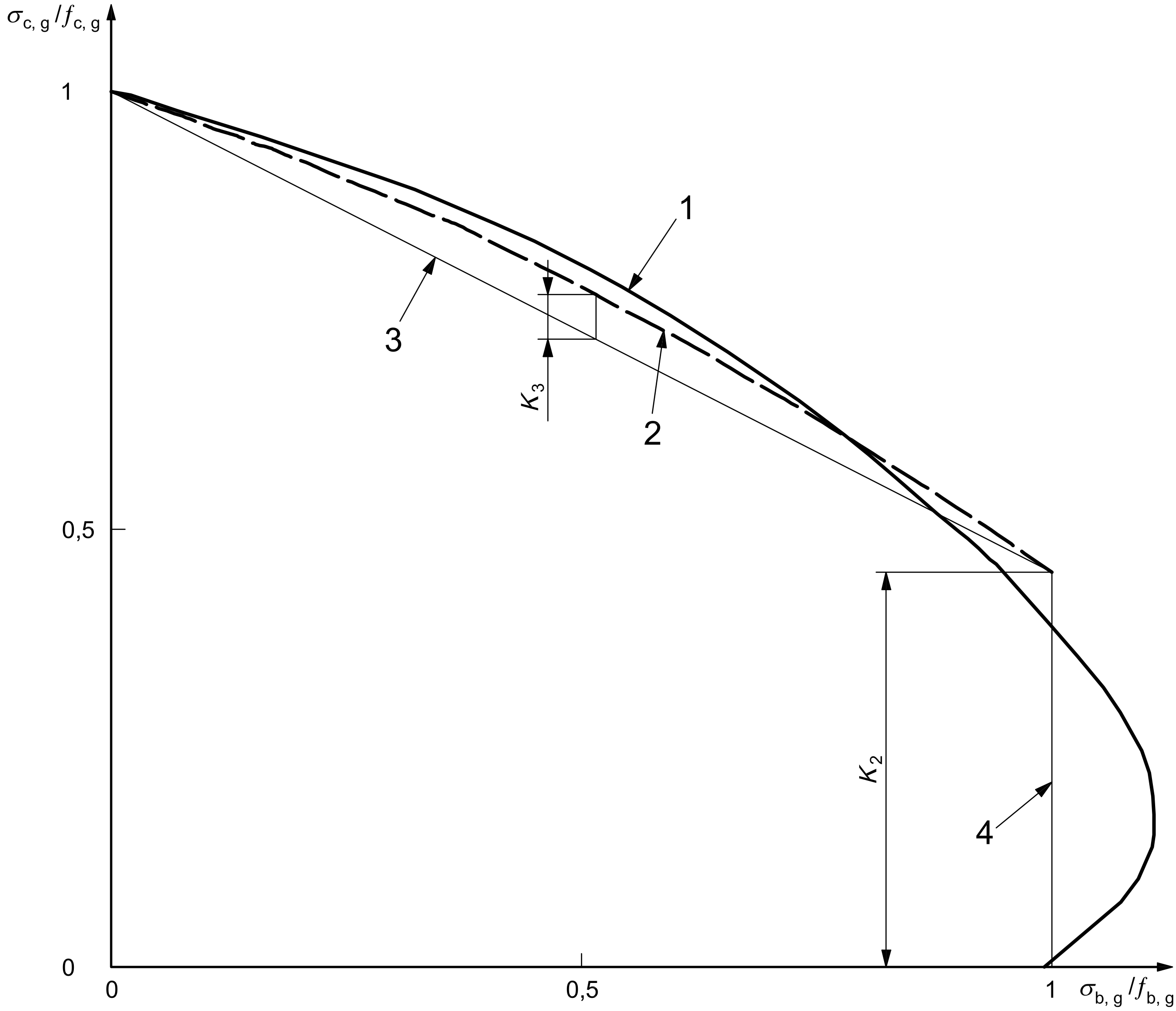
A.13.9.3.1 Axial tension and bending

For a fully grouted, undamaged tubular subjected to combined axial tension and bending forces, there are several ways to estimate its strength. The member may be assessed using Equation (13.3-1) by neglecting the effect of the grout or using Equation (13.9-19) if the maximum stress due to tension is small in comparison to that due to bending. For a fully grouted, dented tubular member, Equations (13.7-24) and (13.7-26) may be used by replacing f_b with $f_{b,g}$.

A.13.9.3.2 Axial compression and bending

The design requirements are based on those given in Reference [A.13.9-9] for concrete-filled tubular members subjected to combined axial compression and bending. The background to the design requirements is documented in References [A.13.9-10] and [A.13.9-11]. The interaction equations for fully concrete-filled tubular members are judged to be applicable to grout-filled tubular members because their behaviour, and the strength contribution of the grout are very similar to those of concrete filled tubulars.

The parameter, K_1, as given in Equations (13.9-28) and (13.9-29), is the non-dimensional axial compressive strength of the member. A slightly different K_1 equation is given in Reference [A.13.9-9]. The significance of the parameters K_2 and K_3 is illustrated in Figure A.13.9-2. The influence of the parameter K_2 is low for tubular members having a small outer diameter and a high slenderness ratio, regardless of the D/t ratio. Low values of K_2 generally also mean a low value of K_3, which can be negative for very small diameters and a high slenderness ratio. In general, the advantageous effect of grouting resulting in increased strength diminishes as the diameter becomes smaller.



Key

1 interaction curve from analytical solution

2 approximating parabolic curve in Equation (13.9-24)

3 Equation (13.9-38) for fully utilized and fully grouted member with $K_3 = 0$

4 Equation (13.9-39) for fully utilized and fully grouted member

$\sigma_{c,g}$ axial compressive stress in fully grouted member

$\sigma_{b,g}$ bending stress in fully grouted member

$f_{c,g}$ representative axial compressive strength of grouted member

$f_{b,g}$ representative bending strength of grouted member

K_2 variable from Equation (13.9-30)

K_3 variable from Equation (13.9-31)

Figure A.13.9-2 — Illustration of relevance of parameters K_2 and K_3

A.14 Strength of tubular joints

A.14.1 General

The requirements of Clause 14 are intended to provide guidance on the design of tubular joints covering the wide range of joint configurations, geometries and action combinations that can be found in practice.

Despite continuing industry-wide efforts, combinations of test data and analytical techniques continue to be used as a basis for design. Guidance on using tests and analyses is contained in 7.7, while Reference [A.14.1-1] provides methods for selecting an appropriate reduction factor for joint strength to account for a small number of data.

A.14.2 Design considerations

A.14.2.1 Materials

All of the empirical strength equations have been based on measured yield strengths. Very few test results have indicated unexpected low strength due to substandard material properties. However, it is recognized that some limits are implied by the database.

Over the period 1994 to 1996, MSL Engineering (UK)[4)] undertook an update of the tubular joint database and guidance through a Joint Industry Project (JIP), see References [A.14.2-1] and [A.14.2-2]. One important change resulting from the MSL work concerns steels with high yield-to-ultimate strength ratios. Many existing codes do not allow the designer to assume more than a ratio of about 0,7 or 2/3. If the ratio exceeds this limit, the chord yield strength is downgraded to the specified ratio of the tensile strength. The MSL work found that the database justified a limit on the ratio of 0,8 for joints with a chord yield strength of up to at least 500 MPa.

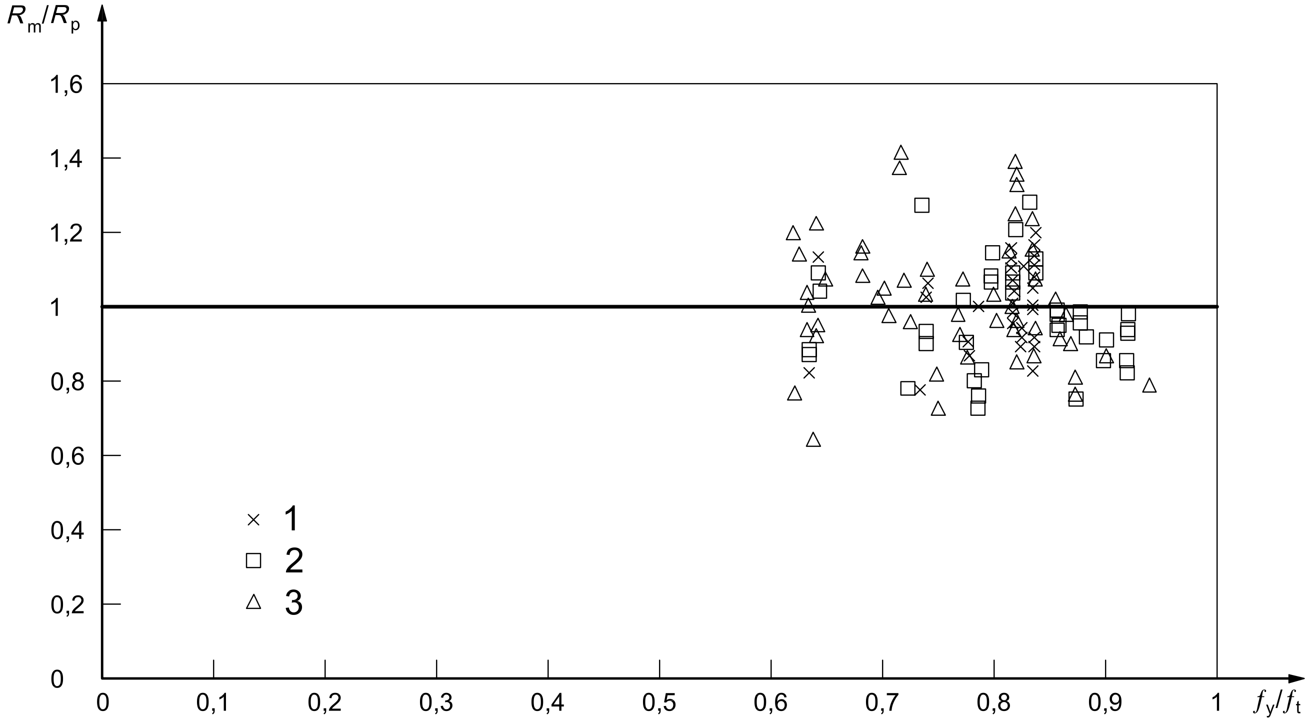
Figure A.14.2-1 shows a plot of the ratio of measured to predicted K-joint strength against the chord material yield-to-ultimate strength ratio. This is based on the updated MSL database [A.14.2-1] with the predicted representative joint strength taken from Clause 14 using actual chord yield strength values and the predicted representative strength suitably upgraded to determine the mean strength of the joint. It can be seen from Figure A.14.2-1 that no effect of yield strength occurs for yield-to-tensile strength ratios up to 0,8. Similar observations have been made with respect to other joint types and load cases. Some steels exhibit adequate toughness even when the ratio exceeds 0,8. However, use of this limit is still considered appropriate because strength equations generally imply some degree of strain hardening. Lack of strain hardening can lead to large local strains and premature cracking. Use of the limiting yield-to-tensile strength ratio of 0,8 ensures that joints exhibit strength and ductility behaviour representative of those in the test database.

The material property range is limited to $f_y \leqslant 500$ MPa. There has, historically, been a concern that the strength of joints with chord yield strength in excess of 500 MPa does not increase in proportion to the yield strength. The concern relates to the potential that the higher yield strength is obtained at the expense of lower ductility and lower strain-hardening capacity, thereby compromising the post-yield reserve strength on which the strength equations rely. This matter is discussed in Reference [A.14.2-3]. A re-evaluation of the test results reported in Reference [A.14.2-3] has been carried out. This re-evaluation has revealed that use of the limiting yield-to-tensile strength ratio of 0,8 is generally adequate to permit the strength equations to be used for joints with 500 MPa $< f_y \leqslant$ 800 MPa, provided adequate ductility can be demonstrated in both the heat affected zone and in the parent material. It should, however, be noted that the test data reported in Reference [A.14.2-3] is limited to a small number of joint types and loading modes with just 11 joints tested in total.

A later JIP [A.14.2-4] investigated the static strength of high strength steel X-joints. The JIP involved the testing of four compression joints (two with a representative yield strength of 355 MPa and one each with 500 MPa and 700 MPa) and three tension joints (one each with a representative yield strength of 355 MPa, 500 MPa and 700 MPa). The findings presented in Reference [A.14.2-4] appear to indicate that all the joints performed satisfactorily in the tests in terms of strength and ductility, confirming the practicality of using higher strength

4) This information is given for the convenience of users of this International Standard and does not constitute an endorsement by ISO.

steels. The results from this limited data identified that a yield-to-tensile strength ratio of 0,8 could be used to estimate the ultimate compression and tension strengths of the joints. However, no detailed assessments were presented for the joints subjected to tension forces in which cracking was observed prior to reaching the ultimate strength.



Key

1 near zero gap/overlap K-joints

2 overlapped K-joints

3 gapped K-joints

f_y yield strength

f_t tensile strength

R_m measured joint strength

R_p predicted joint strength

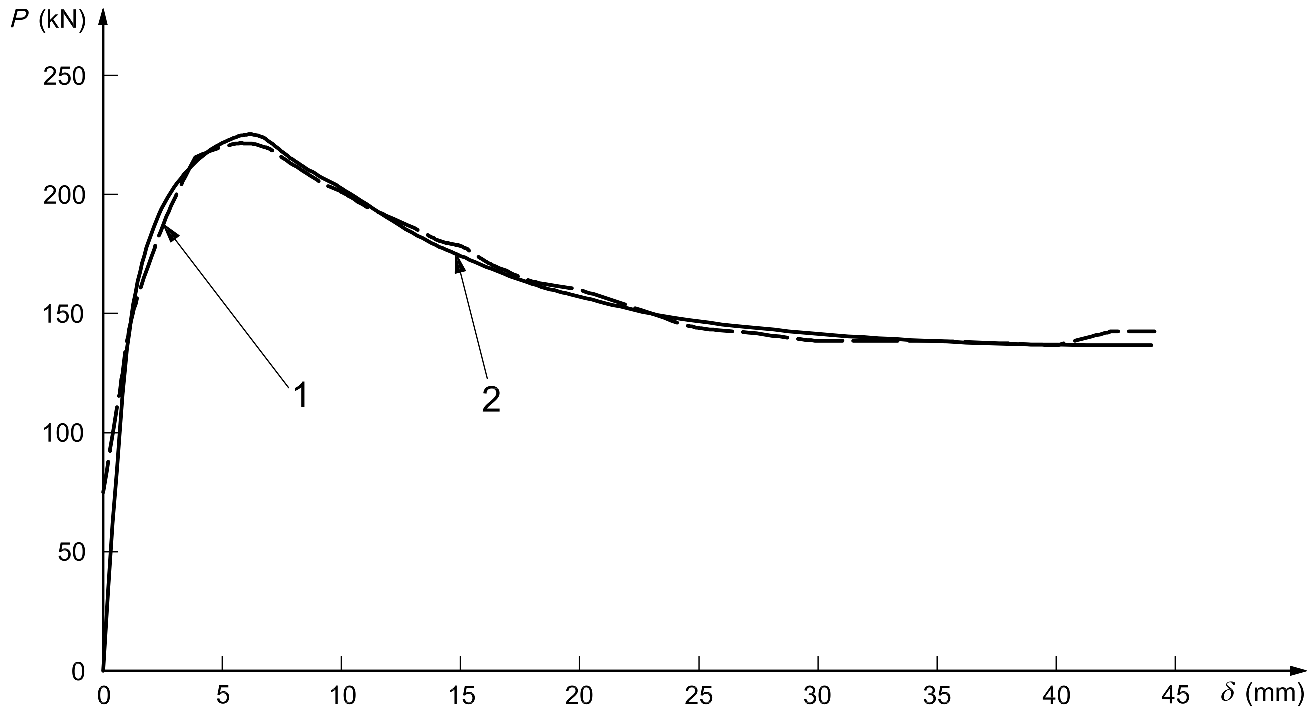
Figure A.14.2-1 — Effect of f_y/f_t for K-joints on joint strength under balanced axial forces

A.14.2.2 Design forces and joint flexibility

Historically, designers of fixed steel offshore structures have usually assumed that the joints are rigid and that the frame can be modelled with members extending to working points at chord centre lines. It has long been recognized, however, that tubular joints can show significant linear elastic flexibility at locations such as skirt pile bracing. Joint flexibility has sometimes been included in calculating fatigue life estimates. Fatigue life estimates for conductor framing connections can sometimes be substantially greater when linear elastic flexibilities are included in the analyses. Full non-linear force-deformation curves for joints can be required for push-over analyses to determine a system's ultimate strength, especially where joint failures participate in the sequence leading to system collapse. Such push-over analyses are common in studies for maintenance and life extension of structures.

In 1993, Buitrago et al [A.14.2-5] published a robust set of equations for the linear elastic flexibility and stiffness of simple tubular joints. Although a number of other sets of formulations are available in the literature, a comparative exercise shows that the Buitrago formulations are wide-ranging, have physical meaning, compare well with hard data, and are simple to use both manually and computationally.

As part of the MSL JIP [A.14.2-1], the understanding of linear elastic flexibility was extended, through analyses of the updated database, and a range of closed-form expressions was established which permit the creation of non-linear force-deformation (P–δ or M–θ) curves, in both loading and unloading regimes, for simple joints across the practical range of load cases and geometries. Details are presented in Reference [A.14.2-6]. Figure A.14.2-2 shows a fitted curve superimposed on the experimental curve.



Key

1 experimental curve

2 fitted curve

P force

δ deformation

Figure A.14.2-2 — Comparison of fitted curve to experimental data for force-deformation behaviour of a simple tubular joint

Reference [A.14.2-7] reports on a pilot study to assess the effect of hydrostatic pressure on tubular joint strength. Double T-joints (DT-joints, see 14.2.4) were studied, and the results indicate that strength can be reduced by up to 30 %, depending on geometry, brace force or moment and hydrostatic pressure. Apart from Reference [A.14.2-7], no other studies on this subject have been identified. Hydrostatic pressure effects can be significant, especially for deep water structures.

A.14.2.3 Minimum strength

Historically, API had a minimum strength requirement that equated to 50 % of the strength of the brace. For earthquake actions, the requirement was essentially 100 % of the brace strength. Aside from earthquake regions, the 50 % strength requirement generally dominated secondary joint design [A.14.2-8] and often took precedence for primary joints. Although it was not intended that this 50 % rule should govern the design of joints it was an inadequate requirement both for accidental situations and for joints critical to the reserve/residual strength of the structure. In general, joint failure prior to member failure is undesirable, due to uncertainty in failure behaviour and the effect on surrounding braces.

The intent in this International Standard is to simplify the requirement for critical load paths. In order to ensure that joints do not fail before members, joint utilization is limited to $1/\gamma_{\mathrm{Z\,j}}$ of the member utilization.

A.14.2.4 Joint classification

Some special cases of the basic planar joint types have their own names. A special case of a Y-joint with a brace angle of approximately 90° is called a T-joint. Another special case of a Y-joint is a double T- or DT-joint, which has braces on opposite sides of the chord, each with a brace angle of around 90°. By its configuration, a DT-joint appears to be a special case of an X-joint with braces under 90° and the through member serving as the chord. However, the axial forces in both braces of a DT-joint are reacted by beam shear in the chord, while in an X-joint the axial brace force is transferred through the chord to the opposite side without a beam shear reaction of the chord. A joint should be classified as a DT-joint if the brace on one side of the chord is in tension and the brace on the other side of the chord is in compression. Both braces should then be treated separately as for a T-joint.

A K-joint where one of the two braces has a brace angle of approximately 90° (see, for example, case a) in Figure 14.2-2) is sometimes referred to as an N-joint.

Joint classification is based on axial force pattern, as well as on joint configuration. The favourable strength factor Q_{u} (see 14.3.3) for K-joints reflects their reduced joint can ovalization as compared to Y-joints. The Q_{u} values for X-joints are typically worse than for Y-joints, because X-joints have an even more severe ovalization tendency.

If the axial force in a given brace corresponds within ± 10 % to one of the standard joint types, it is permissible to classify the brace end as totally of that joint type: no interpolation is required. Many real joints have braces that are not clearly of one given type. In other words, the distribution of forces is complex, in the sense that an individual brace axial force should be divided into portions that are each treated as a Y-, K- or X-joint. Figure 14.2-2 c) and h) contain braces with mixed classifications.

Several schemes for automating the classification process have evolved over the years. None is unique. In all of them, braces belonging to a particular joint type are identified and the geometrical information is catalogued. Braces lying in a common plane and on the same side of the node are identified and the gap between them is computed. Each brace is evaluated based on its axial force for each load case. Classification may change from load case to load case and is often different for each brace at a given joint. Mixed classifications generally occur.

In a typical classification scheme, braces whose axial force component perpendicular to the chord is essentially balanced by axial forces in other braces on the same side of the chord are treated as K-joints. Figure 14.2-2 a), d), e) and g) are such cases, as are the lower braces shown in Figure 14.2-2 c) and h). Braces whose perpendicular force components are reacted by beam shear in the chord are treated as Y-joints, as in Figure 14.2-2 b), even though its geometrical appearance suggests a K-joint. Finally, braces whose perpendicular forces follow neither K-joint nor Y-joint classification are treated as X-joints. In other classification schemes, the hierarchy is K- followed by X-, with T/Y- being the default.

The classification scheme does not quantitatively address multiplanar connections, even though fixed steel offshore structures are generally space frames, not planar trusses. Furthermore, the scheme does not recognize that several braces in a given plane can simultaneously contribute to ovalization of the chord, for example in launch frames and the other examples illustrated in Figure A.14.2-3. Such cases can produce a more adverse force distribution than is recognized in the classification scheme.

An alternative approach to joint classification is to use the ovalization parameter, α, from AWS D1.1 [A.14.2-9]. The attraction of the α-based classification in AWS D1.1 is that it does not require any decisions concerning classification. In a general sense, it encompasses the simple joint classification scheme recommended in this International Standard and provides a promising methodology for multiplanar joints. However, deficiencies exist with the AWS approach where

a) α significantly exceeds baseline X-joint classification,

b) the diameter ratio β is above 0,9, or

c) a low value of α is not the result of classical K-joint action.

AWS D1.1 does not properly incorporate the effect of the gap or address tension failures in the same manner as this International Standard.

Another JIP [A.14.2-10] has generated a considerable database of finite element data for multiplanar joints having no overlapped braces and being subjected to axial brace forces. Refined expressions are given for the ovalization parameter, α, which are used within the AWS D1.1 approach.

In cases such as those illustrated in Figure A.14.2-3, it is conservative to first find the sum of the force components perpendicular to the chord that are passed through the chord section and then assume that the strength is the minimum of any one of the brace-chord intersections when acting as an X-joint. General collapse calculations or finite element analysis can reduce the conservatism.

Additional guidance specific to multiplanar T/Y-, K- and X-joints with axial forces can be found in Reference [A.14.2-11]. More contemporary information on multiplanar T/Y- and K-joints is available in References [A.14.2-12] to [A.14.2-14]; however, the guidance provided in these is not especially robust, and there are general restrictions as to force pattern, as well as to joint configuration.

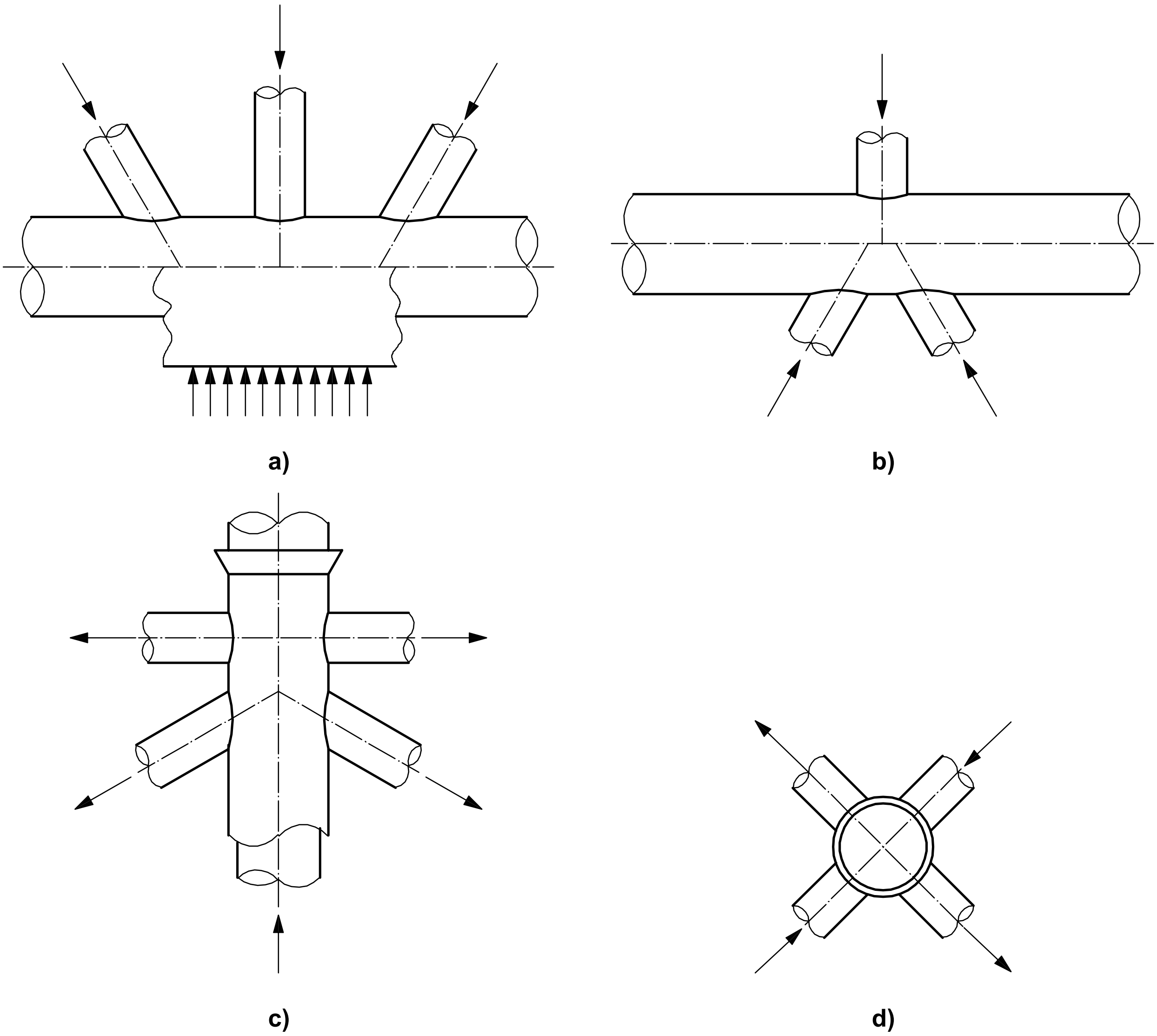


Figure A.14.2-3 — Force patterns causing chord ovalization greater than considered in strength equations

A.14.2.5 Detailing practice

The API guidelines [A.9.1-1] have been changed in this International Standard in several important ways. The can and stub length dimensions are unchanged, but length measurement does not include thickness tapers.

The guidance on overlap dimension has been changed to simplify analysis and make measurement easier during fabrication. The minimum overlap dimension is a recommendation rather than an absolute requirement. There are many practical instances where only minor overlap occurs. These cases are fully amenable to contemporary analysis for both strength and fatigue. Fabrication of small overlaps has not proved particularly difficult; however, any overlap in a joint reduces the extent to which a joint can be inspected.

In many instances, using the minimum chord can lengths provides significantly less strength than can be achieved by lengthening the can. Equation (14.3-11) can be used to optimize the strength of the joint by providing sufficient can length.

A.14.3 Simple circular tubular joints

A.14.3.1 General

A.14.3.1.1 Historical background

The requirements and recommendations contained in this International Standard are based on interpretation of available data, both experimental and numerical. As with all empirically based practice, the recommendations have specific validity ranges, although for this International Standard the qualification has little significance since the ranges cover the wide spectrum of geometries and materials currently used in practice.

Apart from the yield strength limitations given in A.14.2, the recommendations can still be used for joints with geometries which lie outside the validity ranges, by taking the usable strength as the lesser of the strengths calculated on the basis of

— actual geometrical parameters, and

— imposing the limiting value parameters for the validity range, where these limits are infringed.

API requirements [A.9.1-1] included a limit on (punching) shear stress. This limit is unnecessary for joints within the given parametric ranges that have empirical equations assigned to them. However, a limit on shear stress can still be appropriate for cases where the parametric ranges are exceeded or where there are no empirical equations founded on a suitable database. Both experimental and calibrated finite element results can constitute an acceptable database.

Most of the detailed requirements and recommendations provided in API RP2A relate to simple joints consisting of circular hollow sections. Other offshore design documents are based on an early 1980s experimental database. Many data have since been added, often as a consequence of testing a simple joint as a reference in the course of examining a complex configuration. The database for in-plane bending has doubled [A.14.3-1], while, in addition to experimental data, FEA studies have improved the understanding of tubular joint behaviour [A.14.3-1]. Hence, a distinct need existed for a complete review and update of simple joint practice.

The MSL JIP[A.14.2-1], [A.14.2-2], over the period 1994-1996, significantly influenced practice for simple and overlapping joints. The general approach adopted in the MSL JIP was

a) collation of comprehensive databases of worldwide experimental and pertinent FEA results,

b) validation and screening of the databases,

c) fitting curves to the data,

d) comparison of databases and of derived strength formulations with previous practice, and

e) selection of appropriate formulations.

Baseline screening criteria adopted in the MSL JIP were

— chord diameter, and

— gap dimension for K-joints.

A.14.3.1.2 Data range for chord diameter

A lower limit of 100 mm was imposed in collecting data for the validated database. The argument for a cut-off limit stems from a concern that small scale specimens are not representative of fabricated joints found offshore from a material, weld or tolerance standpoint. It is recognized however, that failure modes for tubular joints subjected to predominantly static forces are generally dominated by gross, plastic ovalization of the chord cross-section and, therefore, size effects are expected to be less dominant than for joints subjected to repetitive actions simulating fatigue. In the MSL JIP, no distinct influence of scale down to 100 mm was observed.

A.14.3.1.3 Data range for gap dimension for K-joints

The validated database for K- and T/Y-joints contains both non-overlapped and overlapped geometries. A filter based on the gap dimension was required. One possible criterion was to consider all joints with a gap greater than or equal to zero as non-overlapped. However, this can lead to ambiguity, as some references only record the eccentricity with calculated gaps falling into a narrow range around zero, e.g. ± 1 mm. It was therefore not clear that the zero gap criterion should be strictly applied. From a practical viewpoint, the feasibility of achieving a zero gap is questionable. To overcome these difficulties, it was decided to select non-overlapped joints as those having a finite gap with a small lower limit. Similarly, overlapped joints were defined as those having a finite overlap of at least a specific small amount. Nominally zero gap specimens, g_0 were defined by the following criterion:

$$-g_0 \leqslant g \leqslant g_0 \tag{A.14.3-1}$$

where

g_0 = 7,5 mm or 2 T, whichever is the greater;

T is the thickness of the chord.

Use of the above criterion excluded only a relatively small number of data from either the gapped or overlapped databases.

A.14.3.1.4 Data range for other dimensions

No other limitations on dimensions were imposed. However, the derived non-dimensional quantities were screened as follows.

a) Length/diameter ratio

Specimens having very low ratios of chord length to radius were removed, as they would be unduly affected by short chord effects and by end restraints, and because unusual chord stress flow patterns would exist at the joint. A limit on the ratio of 3 was initially selected, although this limit was tightened following data analyses.

b) Thickness ratio

The vast majority of the data have joints with nominally the same or greater chord wall thickness than that of the brace. This is in accordance with general offshore practice. Some data, however, have relatively thick braces (no doubt an attempt by the investigator to induce joint failure rather than brace failure) with thickness ratios τ as high as 5,5. For nominally the same chord and brace thickness and allowing for a ± 5 % variation of both, an upper limit of $\tau = 1,1$ was selected. Similarly, some investigators have attempted to force joint failure rather than brace failure by reinforcing the brace with stiffeners or by using concrete within the brace. These specimens were also rejected from the database.

A.14.3.1.5 Data range for material properties

All specimens where the representative chord yield strength was not recorded have been excluded from the screened database.

A.14.3.1.6 Utilization of chord and brace

The data were segregated into two groups:

a) joints where nominal stresses in both brace and chord were below yield;

b) joints where nominal stresses in brace or chord have exceeded yield.

The first group was treated as true joint strength data with no reservation. The second group was treated as lower bounds to joint failure.

A total of 1 066 simple joint specimens with D greater than 100 mm were screened. Of these 653 specimens were validated and provided the database. The significance of establishing a suitably screened database cannot be overemphasized. The differences in the various previous/other code requirements on joint strength are partly due to differences in databases. The screened data have been used to assist in the creation of suitable expressions for joint strength, using regression analysis based on minimizing the percentage differences and using statistical calculations, which are characterized by a 95 % survivability level and a 50 % confidence level.

A.14.3.2 Basic joint strength

The basic API format for strength equations has been retained. A change to API practice is that where interpolation of Q_u was previously adequate for braces with mixed classification subjected to axial forces, interpolation is now based on a weighted average of P_uj, since Q_f also varies with axial force classification. Taking Figure 14.2-1 h) as an example, the diagonal brace has a 50 % K- and a 50 % X-joint classification. In this case, P_uj is calculated separately for K-joint classification and for X-joint classification. In the calculation for X-joint classification, any effects of short can lengths are included in accordance with 14.3.5. The representative joint axial strength can thereafter be calculated as follows:

$$P_\text{uj} = 0{,}50\ P_\text{uj,K} + 0{,}50\ P_\text{uj,X} \tag{A.14.3-2}$$

where

P_uj is the representative joint axial strength, in force units;

$P_\text{uj,K}$ is the representative joint axial strength for K-joint classification, in force units;

$P_\text{uj,X}$ is the representative joint axial strength for X-joint classification, in force units.

A.14.3.3 Strength factor, Q_u

The various Q_u factors have been derived from appraisals of screened steel model test data, supplemented by FEA data, where available, for each joint and force type. In deriving the factors, the formulations in other codes were examined and the best formulations for capturing the effects of the joint parameters (e.g. β and γ) were selected and the coefficients were adjusted to give representative strength values. In some cases, new formulations provided significant improvements in the coefficient of variation (COV) or have a wider range of applicability. In particular, the former applies to the formulation for overlapped K-joints with axial forces, while the latter applies to the out-of-plane bending formulation. Full details of the assessments appear in Reference [A.14.2-1].

In the derivation of design formulations for the static strength of tubular joints, it has been traditional practice to fit Q_u functions to empirical data, without accounting for the effects of chord force (now taken into account via the Q_f factor).

Further improvements to the strength factor can be found in References [A.14.3-2] and [A.14.3-3].

A.14.3.4 Chord force factor, Q_f

Compared to previous codes, a substantial change to the chord force factor (Q_f) is made in this International Standard — at least in respect of T/Y- and K-joints subjected to axial forces. This change follows an exhaustive appraisal of the effect of chord forces on joint strength [A.14.2-1].

Inspection of the Q_f factor shows that no dependency on γ is given. Previous codes included a dependency which was based on forcing DT-joints of one specific γ and K-joints of another specific γ to align. The new appraisals have indicated that any γ dependency in K-joints is small and its influence is therefore not included in the requirements in this International Standard.

Comparison of the Q_f formulation in this International Standard with those of existing guidance such as API shows the following differences.

The dependency of Q_f on γ has been removed.

The factor $C_1 = 14$ for K-joints in Table 14.3-2 has been obtained from an examination of data from both tests for which chord axial forces are only in equilibrium with brace forces and tests in which additional chord forces were applied.

For T-joints with axial brace forces, the effect of chord in-plane bending has been relaxed. Substantial in-plane bending moments can be generated by axial brace forces in T-joints. These chord moments can be expected to affect joint strength, as confirmed in finite element analyses by van de Valk [A.14.3-4]. However, due to chord length and end plate effects and inherent experimental scatter, doubts over true chord boundary conditions exist. The extraction of a chord stress effect from the FEA data proved most difficult. There is a strong correlation between the chord in-plane bending and joint failure with β, with the effect of high chord in-plane bending being captured by the β term in the Q_u function. Therefore, the Q_f effect is hidden in the Q_u function. Recognizing that chord in-plane-bending moments in structures can be higher than those attained in tests, an allowance for Q_f was made. This was achieved by assuming that Q_f reaches a value of 0,8 when chord in-plane-bending reaches the yield moment.

It was recognized in the MSL JIP [A.14.2-1] that chord forces are one of the least understood parameters affecting joint strength, there being little data to properly define Q_f. Recent work by EWI/API [A.14.3-5] has continued to develop alternative formulations. To date, the findings from a comprehensive study for DT- and X-joints subjected to brace compression or brace in-plane bending on the effects of chord forces, including axial, in-plane bending and out-of-plane bending, have been published [A.14.3-6]. In light of the EWI/API work, the coefficients for X-joints given in Table 14.3-2 were adjusted to bring Q_f more in line with the EWI findings.

Further improvements to the chord force factor can be found in References [A.14.3-2] and [A.14.3-3].

A.14.3.5 Y- and X-joints with chord cans

The reduced strength for simple Y- and X-joints with axial forces and short can lengths is supported by numerical and experimental data, see Reference [A.14.2-1]. No reduction in strength is required for K-joints with axial forces. The lack of data has precluded an assessment of strength reduction (if any) for joints subjected to moments or for complex joints.

The API provisions for force transfer across chords have been extended to cover T-joints with axial forces. X-joints with axial forces and with $\beta > 0,9$ increasingly transfer force across the chord through membrane action, and this beneficial mechanism is recognized. The requirements of this International Standard are also intended for application to other cases where force transfer through chords occurs, e.g. launch frame joints.

A.14.3.6 Strength check

The interaction of axial force and bending moments on joint performance is evaluated using an interaction equation which represents a change from the trigonometric equations that have historically existed in API and other codes. However, Equation (14.3-12) is identical to that already in use in the UK [A.14.3-7] and is supported by experimental studies carried out at the University of Texas in the mid-1980s, see References [A.14.3-8] and [A.14.3-9]. Although there is little improvement in reliability between the present

Equation (14.3-12) over that of API when the utilization is less than 1,0, the present equation is more sensible when the utilization exceeds 1,0, as can be found in assessments. An improved understanding is beneficial when deciding on any remedial measures for overstressed existing structures.

Although the discussion in A.14.3 describes the changes that have been made to offshore practice for simple joints, in most instances the overall impact is limited. MSL Engineering has made a brief study of the amalgamated impact of the new guidance on a four leg structure in the central North Sea [A.14.2-1]. Only the changes in simple joint design guidance were considered. The effect of the new guidance on the design was modest, although the general perception is that overall reliability has improved significantly.

A.14.4 Overlapping circular tubular joints

Guidance on the strength of overlapping joints has existed in API [A.9.1-1], HSE [A.14.3-7], CIDECT [A.14.2-11], and other practices for more than a decade. However, previous guidance has never addressed the effects from moments or from out-of-plane overlap. Recent work[A.14.4-1] to [A.14.4-6] has shown that the previous guidance for the axial force strength of joints overlapping in-plane is inadequate. A relatively complete summary of the problems with the previous guidance and the background database can be found in Reference [A.14.4-4].

The guidance in this International Standard has been based on MSL's JIP results [A.14.2-1]. Pertinent test evidence of calibrated FEA results may be applied to specific cases following the principles in 7.7.

In several respects, the guidance in this International Standard is simplified compared with the API and HSE guidance. For example, there is no longer a need to routinely calculate weld lengths. However, in more precise analyses such lengths can still be necessary. Reference [A.14.4-5] reproduces equations for these calculations.

The guidance expands the MSL JIP recommendations into a set of considerations that should avoid the need for FEA in all but the most unusual or failure-critical cases. There are simple, but conservative suggestions for addressing both in-plane and out-of-plane force situations, as well as out-of-plane overlap situations, which are not uncommon offshore. The hope is that ongoing research using FEA (see References [A.14.4-5] and [A.14.4-6]), will eventually help future requirements to be more definitive.

A.14.5 Grouted circular tubular joints

Grouted joints are becoming more common in new steel frame structures and joint grouting is generally a cost-effective means of strengthening older structures. It is now possible to provide guidance [A.14.5-1] based upon engineering approximations and some experimental evidence, see References [A.14.5-2] to [A.14.5-6]. The experimental evidence is primarily for double-skin joints subjected to axial brace forces. However, a JIP by MSL and TNO[A.14.5-7], provides additional data for fully grouted joints, especially those subjected to brace bending moments.

Q_u values for grouted joints, in accordance with the original API guidance, have been derived for Y-, X- and K-joints [A.14.5-1], [A.14.5-3], and are reproduced in Table A.14.5-1.

Table A.14.5-1 — Q_u values for grouted joints

Brace force[a]	Q_u
Axial tension	2,5 $\beta\gamma K_a$
In-plane bending	1,5 $\beta\gamma$
Out-of-plane bending	1,5 $\beta\gamma$
where $K_a = (1 + 1/\sin\theta)/2$	

[a] No term is provided for axial compression, since most grouted joints cannot fail under compression because the compression strength is limited by that of the brace.

Special joint strength investigation can be warranted when braces are grouted, whether or not they frame into grouted chords. Although joint strength is heavily dependent on chord parameters, grouting a brace can reduce its effective diameter which, in turn, reduces β. For joints subjected to moments, the strength in many instances is expected to be limited by the local buckling strength of the brace, even if the brace is compact. Unless evidence to the contrary exists, strength for grouted joints subjected to bending moments should be limited to the plastic moment design strength of the brace.

For double-skin joints, a further limiting strength has been introduced to cater for the potential of failure due to chord ovalization. In such cases, the design strength is the lesser of

a) brace strength,

b) strength calculated on the basis of effective thickness, and

c) strength calculated on the basis of Q_u values for grouted joints.

Consideration of the effects of grouted joints should include review and perhaps revision of the structural model used to determine the sectional forces for the joint. Presence of grout clearly stiffens the joint so that the most appropriate model is likely to be one with a rigid offset from the chord centreline to the chord wall at each incoming brace. If the structure has been modelled with rigid joints located at the chord centreline, the conservatism, or otherwise, of the internal forces should be assessed. If joint flexibility has been introduced at the chord surface, while using a rigid offset to that point, only the flexibility needs to be altered. It is generally conservative to assume grouted joints to have no local flexibility, i.e. that they are assumed to be rigid up to failure.

A.14.6 Ring stiffened circular tubular joints

Some studies on the strength of ring-stiffened joints are given in References [A.14.6-1] to [A.14.6-3]. There are no published ultimate strength results of such joints, apart from those generated in MSL's JIP [A.14.2-1] and in recent studies at EWI [A.14.6-4]. Future data from EWI could assist in providing further guidance in the design of ring-stiffened joints.

Since robust, codified design practices are not yet available, ring-stiffened joints require more engineering attention than many of the simpler joint types. Consequently such joint designs are often more conservative than would be required using test results or calibrated FEA results.

At least three approaches exist for determining the required number and sizes of stiffeners. In all three, the first step is to assume ring dimensions, while being careful to avoid the possibility of local buckling. Then the required number of rings is evaluated. If the number is excessive, the stiffening ring geometry is modified, possibly including the addition of an inner edge flange, and the number of required stiffeners is recalculated. The three approaches are described below.

a) The joint forces are assumed to be fully resisted by the rings assuming elastic behaviour. The ring cross-sectional properties are calculated using an effective flange width from the chord can. The effective flange width and the elastic analysis of the ring are based upon Roark's formulas [A.14.6-5]. Usually, a partial resistance factor on yield strength is applied.

b) The joint forces are assumed to be fully resisted by the rings assuming plastic behaviour. An effective flange width is assumed, and this value is often the same as in approach a). Based upon a simple interaction expression for axial force, shear and moment in the ring, a ring ultimate strength is derived. This strength is downgraded by a partial resistance factor that is normally assumed to be the same as for simple joints.

c) The joint forces are assumed to be resisted by a summation of simple joint strength and ultimate strength behaviour of the rings. The residual ultimate ring strength may simply be calculated as being the shear strength of two cross-sections of the ring. Partial resistance factors are applied to both the simple joint and ring strengths.

Several questions can arise with all of the above methods. It is not always clear how to address brace moments. The usual approach is to break them into couples and take the absolute sum of the forces from axial and moment couple as the applied action. Rings that are outside any brace footprint are also not addressed by any of the approaches listed above.

Although outside rings offer little advantage with respect to fatigue (by reducing stress concentration factors), they can be much more effective than internal rings where ultimate strength is of concern. Often, external rings can be assumed to be fully effective if the clear distance from the edge of a given brace does not exceed $D/2$, although the shear transfer strength of the chord wall between the brace and outer ring should still be examined. The effectiveness of rings under a given footprint is normally assumed to be limited to the particular brace involved. The $D/2$ limit is only of concern for rings at the end of the chord can.

A more general procedure for the design of ring-stiffened joints is to consider sections or planes through the joint and ensure that the strength of all parts severed by the plane is sufficient to resist the applied forces. This approach is quite general, although difficult to automate. Its advantages are that it can address even the most complex conditions, and it often provides a better physical feel for load paths. This approach is recommended as a manual check of expected behaviour whenever possible. However, this approach does not implicitly address potential local buckling or premature cracking.

As with grouted joints, use of ring-stiffened joints warrants a review of the structural model used to determine the sectional forces for the joint. Rings often increase the joint stiffness substantially, such that rigid offsets to the chord surface are appropriate.

A.14.7 Other circular joint types

A general approach based upon material strength principles can be suggested. The potential for local buckling or premature cracking should be investigated. Further information on circular joints with doubler or collar plates can be found in References [A.14.7-1] to [A.14.7-4].

A.14.8 Damaged joints

Reduction in axial or moment strengths may be estimated by taking into account the reduced area or section modulus due to the presence of cracks. References [A.14.8-1] to [A.14.8-3] cover some of the research carried out in this area.

A.14.9 Non-circular joints

Detailed design practices and guidance for some connections with noncircular chords and braces are available in References [A.14.9-1] and [A.14.9-2]. These were developed for on-shore use. Care should be taken with the use of these documents to ensure consistency with the design approach and philosophy in this International Standard.

A.14.10 Cast joints

No guidance is offered.

A.15 Strength and fatigue resistance of other structural components

A.15.1 Grouted connections

A.15.1.1 General

No guidance is offered.

A.15.1.2 Detailing requirements

No guidance is offered.

A.15.1.3 Axial force

No guidance is offered.

A.15.1.4 Interface transfer stress

No guidance is offered.

A.15.1.5 Interface transfer strength

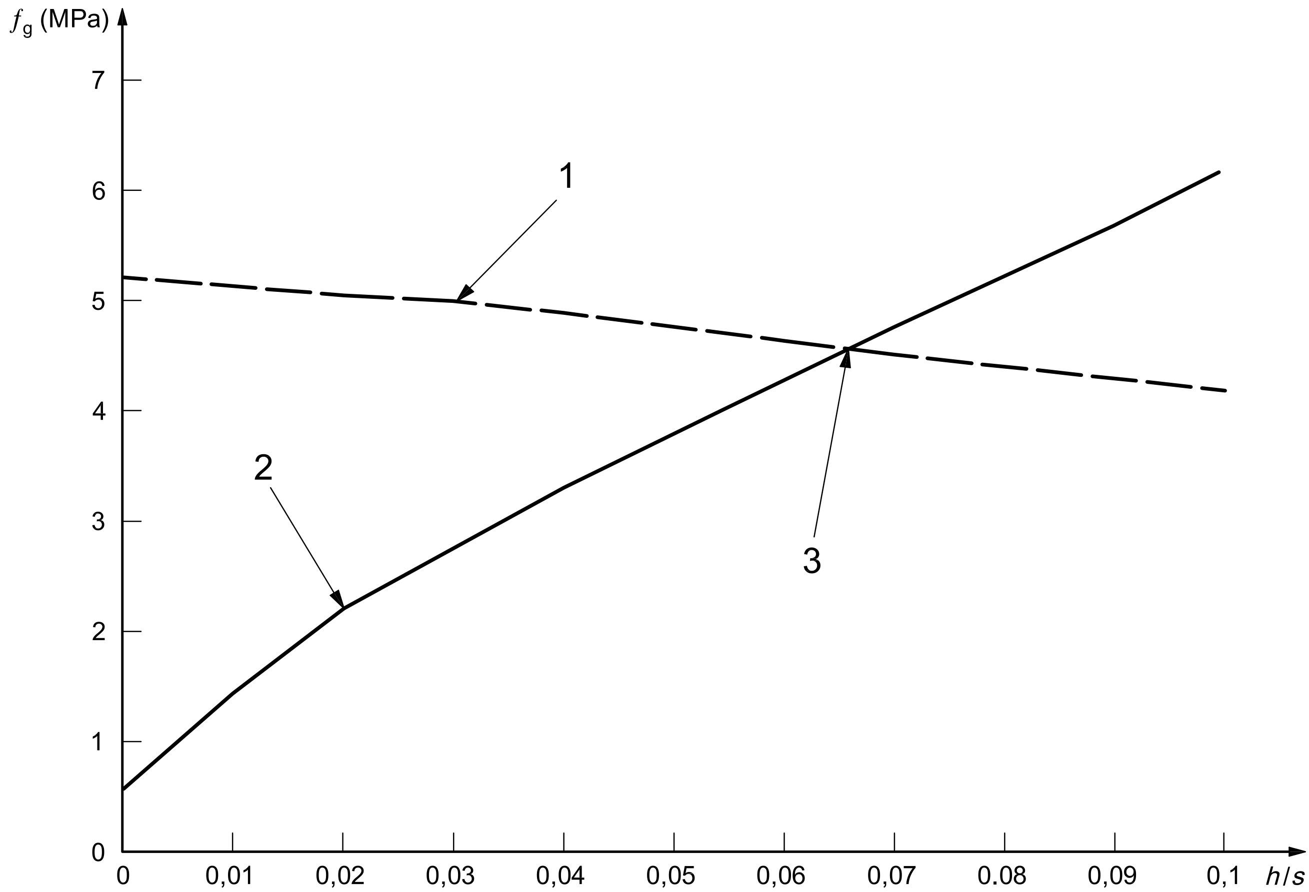
A.15.1.5.1 General

The strength of a grouted connection is defined as the lesser of the following:

a) interface strength — which is the mechanical interlocking strength of the pile to grout interface to avoid sliding;

b) grout matrix strength — which is the shear strength of the grout in the annulus.

An example case showing these two strengths for $f_{cu} = 50$ MPa and $K = 0{,}015$ is presented in Figure A.15.1-1. Effects of movements during grout setting (also known as early age movements) are not included.



Key

1 grout matrix strength ($f_{g,shear}$)
2 interface strength ($f_{g,sliding}$)
3 point of optimal interface transfer strength
f_g representative interface transfer strength
h shear key height
s centre-to-centre spacing of shear keys

Figure A.15.1-1 — Example of grouted connection interface transfer strength

The representative interface strengths for the two failure modes are defined by Equations (15.1-5) and (15.1-6), respectively. For a given grout strength (f_{cu}) and radial stiffness factor (K), there will be an optimum value for the ratio of shear key height to spacing (h/s), as shown at the point where the two lines cross in Figure A.15.1-1. For most practical designs this optimum strength will be achieved for a ratio of h/s in the region of 0,05 to 0,07, when the early age degradation effect is included. To limit the effects of movements during grout setting and degradation due to fatigue, as described in A.15.1.5.3 and A.15.1.7, respectively, the optimum h/s value should not be exceeded.

A.15.1.5.2 Ranges of validity

For a grout strength less than 20 MPa, as can be encountered during the early stage of curing, Equations (15.1-5) and (15.1-6) may be used down to a lower limit of $f_{cu} = 7$ MPa, provided that the partial resistance factor $\gamma_{R,g}$ is increased from 2,0 to 2,5.

A.15.1.5.3 Effect of movements during grout setting

The background to the development of the design requirements is detailed in Reference [A.15.1-1]. The major area of uncertainty in the design of grouted connections is the potential degradation of both strength and fatigue performance due to relative movements between the inner tubular member (the pile) and the outer tubular member (the sleeve) during the grout setting period (generally the 24 h period following grout placement). This movement is often termed “early age cycling”. The exact nature of the degradation and the parameters which control it are not fully understood. However, it is known that strength degradation due to early age cycling increases with h/s.

During the grouting operation, and during grout setting, the potential exists for the inner and outer steel tubulars to move relative to each other due to environmental action on the structure, particularly when the tubulars are a foundation pile and its sleeve. Tests to simulate this effect in the laboratory have demonstrated the following consequences.

a) The reduction in static strength increases as

- the relative pile-sleeve movement increases,
- the pile shear key density (h/s) increases, and/or
- the grout strength (f_{cu}) increases.

b) The fatigue strength is degraded in direct proportion to the degradation in static strength.

The majority of the laboratory tests were performed with an initial relative movement of $\pm$ 0,35 % of the inner tubular (pile) diameter (D_p). The degradation factors given in 15.1.5.3 are based on these data. This level of movement was chosen based on calculations of the anticipated movements between foundation piles and sleeves for a number of structures, founded on different soils, in sea states typically required for installation activities. If movements in excess of 0,35 % of D_p are anticipated during the grouting and grout setting period, test data will be required to validate the design. As an alternative, pile grippers or other mechanical locking devices can be used to eliminate or reduce relative movements.

In addition, test data on movements during grouting are only available for connections with an h/s ratio up to 0,06. Designs exceeding this value will require either that movements are eliminated or validation tests to quantify the appropriate degradation factor. However, as noted in A.15.1.5.1, efficient designs are possible without exceeding $h/s = 0,06$.

A.15.1.6 Strength check

No guidance is offered.

A.15.1.7 Fatigue assessment

Fatigue testing of grouted connections has demonstrated the following fatigue characteristics:

a) plain grouted tubular connections (without shear keys) do not suffer fatigue damage unless the applied variable force amplitude approaches the ultimate strength of the connection;

b) the fatigue performance of shear key connections increases rapidly as the R ratio of minimum to maximum variable force amplitude moves from -1 (reverse direction cycling of equal magnitude) to 0 (single direction cycling);

c) fatigue damage is characterized by a shouldered force-displacement hysteresis loop, where the shoulder, i.e. the point of initiation of rapid slip, has been shown to correspond to the plain grouted tubular strength of the connection.

The combination of these three observed characteristics indicates that fatigue damage can be averted by ensuring that the peak reverse force cycle is less than the plain grouted tubular strength. Under these circumstances, fatigue damage will not accumulate as the rapid slip mechanism, producing the impact damage which ultimately fails grouted connections in reverse cycling, is averted. Therefore, if the connection's effective grouted length is such that the plain grouted tubular strength exceeds the force amplitude in reverse cycling, fatigue damage will not accumulate by this observed mechanism. If the plain grouted tubular strength is not sufficient, the connection length should be increased. An upper limit on the increase in the connection's effective grouted length by a factor of 1,5 is suggested when used with the partial action and resistance factors given in this International Standard, as there is evidence of an endurance limit in the data with $R = -1$ at an σ_p/f_g ratio of 0,18.

If it is not possible to increase the connection length to provide the necessary plain grouted tubular strength, a detailed fatigue evaluation should be carried out. Reference [A.15.1-1] provides further information on this subject.

It has been observed that the fatigue performance of grouted connections is reduced by movements during grout setting as the h/s ratio increases (see A.15.1.5.3). Therefore, it is advisable to minimize, as far as is practicable, both the h/s ratio and the early age movement. For most practical designs, a structurally efficient pile-to-sleeve connection, with an L_e/D_p of approximately 5 to 6, can be achieved with an h/s of less than 0,04, using a grout with a 28 day strength of 50 MPa. Such an approach is recommended.

A.15.2 Mechanical connections

A.15.2.1 Types of mechanical connectors

No guidance is offered.

A.15.2.2 Design requirements

A.15.2.2.1 General

No guidance is offered.

A.15.2.2.2 Static strength requirements

If connectors are designed to be stronger than the weakest of the connected tubular members, strain hardening effects may be relied upon to ensure that the connector has sufficient strength to mobilize the ductile deformation capacity of the tubular members. The factor by which the connector strength should exceed that of the weakest tubular member depends on the uncertainty of the analysis procedure, the connector material strength, and the consequence of connector failure.

The connector, if not stronger than the weakest of the connected tubular members, should have sufficient ductility to sustain all actions due to deformations, as well as accidental actions, without impairing the safety of the structure.

A.15.2.2.3 Fatigue performance requirements

No guidance is offered.

A.15.2.2.4 Functional requirements

No guidance is offered.

A.15.2.3 Actions and forces on the connector

When the connector can be considered to be axi-symmetric and the connected tubular is subjected to both an axial force (P) and a bending moment (M), the moment can be converted into an equivalent axial force (P_{eq}). The concept of the equivalent axial force is simple and slightly conservative for relatively thin tubulars ($D/t > 20$). The design axial force (P_d) can be expressed as in Equation (A.15.2-1):

$$P_d = P + P_{eq} \tag{A.15.2-1}$$

with

$$P_{eq} = \frac{M(D-t)}{2I} \cdot A = M \cdot \frac{32t(D-t)^2}{D^4 - (D-2t)^4} \tag{A.15.2-2}$$

where

P_{eq} is the equivalent axial force representing the bending moment;

M is the bending moment in the tubular member;

I is the line moment of inertia of the cross-section of the tubular member;

A is the cross-sectional area of the tubular member;

D is the outside diameter of the tubular member;

t is the wall thickness of the tubular member.

A.15.2.4 Resistance of the connector

Non-linear finite element analysis (FEA) can be used to determine the global force-deformation behaviour of the connector. In this case, the connector's ultimate strength can be established by progressing the analysis until the connector can no longer sustain the force, or significant plastic deformation takes place that impairs the functional requirements of the connector, or precipitates its disengagement. In conducting these analyses, due consideration should be given to the connector material properties in terms of yield, strain hardening, ductility and ultimate strength. However, both the connector modelling and the solution methodology should be calibrated to experimental data, see 7.7.

In developing the model for the FEA, the modelling of each adjoining tubular member should extend at least four diameters. For non-linear analysis, a force-displacement diagram should be developed from which the connector resistance can be obtained. The force should correspond to the equivalent axial force on the nominal tubular member, whereas the displacement is the elongation of the connector measured between the two circumferential welds joining the connector to the tubular members. To achieve this, the connector portion of the model should include the non-linear modelling aspects, while the portion of the model corresponding to the nominal tubular, extending beyond each circumferential weld, is linear elastic.

A.15.2.5 Strength criteria

A.15.2.5.1 General

FEA may be used to calculate the stresses induced in the connector for the purpose of strength checks. Strength checks may use linear analysis to determine stress component distributions across particular cross-sections. The stresses induced in connectors by the factored actions can be classified as primary or secondary.

Primary stresses are normal and shear stress distributions across a connector section that result from the application of external actions and are required to satisfy equilibrium. Primary stresses are not self-limiting and, if not controlled, will induce failure. Stresses arising from axial force and bending moments acting in the tubular members are primary stresses.

Secondary stresses are normal and shear stress distributions across a connector section that are self-limiting and self-equilibrating. Stresses resulting from assembly of connectors can be primary or secondary stresses.

Other stresses are pure shear stresses and bearing stresses.

A pure shear stress is an average stress over a connector cross-section that is parallel to the direction of the acting force. An example of this stress is the average stress along the thread roots due to an axial force on the connector.

A bearing stress is an average stress normal to contact surfaces between connector parts mutually or between a connector part and the connector body, such as those between engaged threads, at preload shoulders, or between washers, dogs, etc.

Connector design using stresses resulting from linear FEA is based on criteria from ASME [A.15.2-1]. In order to take advantage of ductile steels and eliminate stress limits based on ultimate stress, material for connectors should

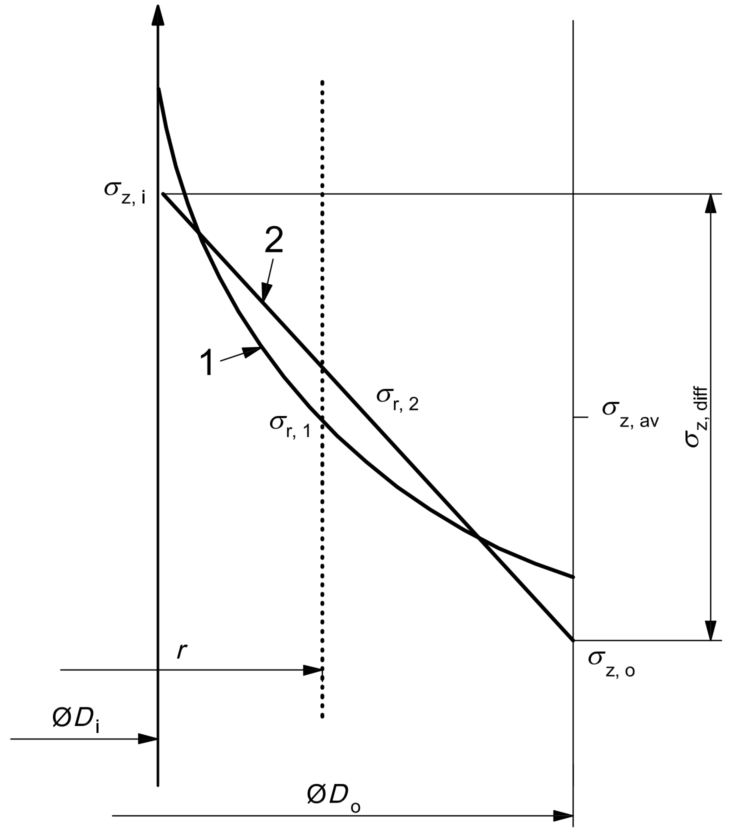
a) have at least 20 % elongation,

b) have ratios of yield-to-ultimate stress no greater than 0,75, and

c) be sufficiently tough at the lowest anticipated service temperatures.

A.15.2.5.2 Linearization and Von Mises stresses

In order to use the stress criteria given in A.15.2.5.3 to A.15.2.5.6, each of the stress component distributions across the section of interest should first be linearized, from which an average and a differential stress can be obtained, as shown in Figure A.15.2-1.



$\sigma_{z,i}$ and $\sigma_{z,o}$ are determined by satisfying

$$\sum_{r=D_i/2}^{D_o/2} \sigma_{r,1}\ \mathrm{d}r = \sum_{r=D_i/2}^{D_o/2} \sigma_{r,2}\ \mathrm{d}r$$

and

$$\sum_{r=D_i/2}^{D_o/2} \left(\sigma_{r,1} \times r^2\right) \mathrm{d}r = \sum_{r=D_i/2}^{D_o/2} \left(\sigma_{r,2} \times r^2\right) \mathrm{d}r$$

Average and differential stresses are

$$\sigma_{z,av} = 0{,}5\ (\sigma_{z,i} + \sigma_{z,o})$$

$$\sigma_{z,diff} = 0{,}5\ (\sigma_{z,i} - \sigma_{z,o})$$

Key

1 elastic stress distribution
2 linearized stress distribution
$\sigma_{z,i}$ axial stress at inner face
$\sigma_{z,o}$ axial stress at outer face
r radius to point being considered
D_i inner diameter
D_o outer diameter
$\sigma_{r,1}$ stress at radius r for elastic stress distribution
$\sigma_{r,2}$ stress at radius r for linearized stress distribution
$\sigma_{z,az}$ average linearized axial stress
$\sigma_{z,diff}$ difference between average and maximum linearized axial stresses

Figure A.15.2-1 — Linearization of axial stress components

The Von Mises effective stress can be calculated using Equation (A.15.2-3):

$$\sigma_e = 0{,}707\left[\left(\sigma_r - \sigma_\theta\right)^2 + \left(\sigma_\theta - \sigma_z\right)^2 + \left(\sigma_z - \sigma_r\right)^2 + 6\left(\tau_{r\theta}^2 + \tau_{\theta z}^2 + \tau_{zr}^2\right)\right]^{1/2} \tag{A.15.2-3}$$

where

σ_e	is the Von Mises effective stress;
σ_r	is the normal radial stress;
σ_θ	is the normal hoop stress;
σ_z	is the normal axial stress;
$\tau_{r\theta}$, $\tau_{\theta z}$, τ_{zr}	are the shear stress components.

The average Von Mises stresses ($\sigma_{e,av}$) can be calculated from the average values of the linearized stresses and the maximum Von Mises stresses ($\sigma_{e,max}$) can be calculated from the maximum values of the linearized stresses, taken as the average +/– the differential stress.

If detailed radial and hoop stresses are not available, then the radial stress at the internal wall, $\sigma_{r,I}$, may be assumed to equal the internal pressure, $\sigma_{r,o}$ may be assumed to equal the external pressure and the hoop stresses may be assumed to equal half the radial pressures.

The intent of linearization is

a) to eliminate the effect of the peak stresses arising from elastic FEA at the location of stress raisers, and

b) to account for the global section behaviour.

Linearization involves finding, for each stress component distribution across the thickness of the section, a linear distribution whose equivalent force and moment are the same as those of the actual distribution (see Figure A.15.2-1). The average and differential components of the normal axial stress distribution physically correspond to the membrane and bending components acting on that section in accordance with P/A and $M\,(D - t)/2\,I$.

Linearization of the shear stress is not recommended. Instead, an average value across the section suffices for the calculation of the effective Von Mises stresses.

In cases where torsion is a significant external action, the average shear stress ($\tau_{r\theta}$) at the cross-section of interest should be approximated by the shear stress calculated by $T\,r/J$, where T is the applied torque, r is the mean radius of the section, and J is the polar moment of the section.

Strength checks should be made at both connector cross-sections and through other sections. For instance, in flanges cylindrical sections parallel to the flange axis and containing the bolt circle should be checked. For arbitrary sections, the stress components are rotated from the global coordinates to the local coordinates parallel and perpendicular to the section in question. The hoop component, however, remains the same for axi-symmetric models.

A.15.2.5.3 Primary stress criteria

For primary stress checks, the portion of the stress due to secondary forces, such as preload due to assembly, may be subtracted from the total stress calculated due to actions and forces following preloading.

The primary average Von Mises effective stress, $\sigma_{e,av}$, and the maximum Von Mises effective stress, $\sigma_{e,max}$, resulting from factored actions should satisfy both the following conditions:

$$\sigma_{e,av} \leqslant \frac{f_y}{\gamma_{R,t}} \tag{A.15.2-4}$$

$$\sigma_{e,max} \leqslant \frac{f_y}{\gamma_{R,b}} \tag{A.15.2-5}$$

where, in addition to the definitions in A.15.2.5.2,

f_y is the representative yield strength of the connector material;

$\gamma_{R,t}$ is the partial resistance factor for tension; $\gamma_{R,t}$ = 1,35 for cases that are dominated by environmental action, $\gamma_{R,t}$ = 1,6 for cases where environmental actions are not included;

$\gamma_{R,b}$ is the partial resistance factor for bending; $\gamma_{R,b}$ = 0,9 for cases that are dominated by environmental action, $\gamma_{R,b}$ = 1,1 for cases where environmental actions are not included.

Strains developed under primary stress conditions should also be limited so that the connector's sealability, its ability to disassemble, and/or the function of the tubular string are not adversely affected.

A.15.2.5.4 Primary plus secondary stress criteria

The average plus differential Von Mises effective stress, $\sigma_{e,av + diff}$, as calculated by linear analysis under preload and factored actions, should additionally satisfy the following condition:

$$\sigma_{e,max} \leqslant \frac{f_y}{\gamma_{R,b}} \tag{A.15.2-6}$$

where $\gamma_{R,b}$ is the partial resistance factor for bending; $\gamma_{R,b} = 0,5$ for cases that are dominated by environmental action, $\gamma_{R,b} = 0,6$ for cases where environmental actions are not included.

A.15.2.5.5 Shear stress criteria

The average shear stress on a gross section, τ_{av}, resulting from factored actions should satisfy the following condition:

$$\tau_{av} \leqslant \frac{f_y}{\gamma_{R,v}\sqrt{3}} \tag{A.15.2-7}$$

where further to previous definitions, $\gamma_{R,v}$, the partial resistance factor for shear stresses, = 1,25.

A.15.2.5.6 Bearing stress criteria

The average bearing stress, σ_{br}, from factored actions should satisfy the following condition:

$$\sigma_{br} \leqslant \frac{f_y}{\gamma_{R,br}} \tag{A.15.2-8}$$

where further to previous definitions

σ_{br} is the bearing stress;

$\gamma_{R,br}$ is the partial resistance factor for bearing stresses = 1,0.

A.15.2.6 Fatigue criteria

A.15.2.6.1 General

In order to be able to predict the fatigue life at a particular location of a structural component, both the stress history at the point under consideration and the resistance of the material to time-varying stresses need to be determined.

For the S–N curve method, the stress history during the lifetime may be given in statistical form as the long-term stress range distribution(s) at the most highly stressed location(s) of the connector. The stress ranges at the location(s) of interest may be determined by linear FEA of a suitable structural model, using all forces and moments occurring in the tubular member as input. The resistance of the connector material should be given by an appropriate S–N curve. If such an S–N curve is not known, it should be determined experimentally. The damage due to each stress range is the number of times, n, that the stress range, S, occurs in the long-term stress range distribution, divided by the allowable number of cycles to failure, N, from the S–N curve. Total damage is obtained by the linear accumulation of the damage done by each stress range, in accordance with Palmgren-Miners rule. The method is described in detail in Clause 16. In the calculations for mechanical connections, the effect of the mean stress due to make-up and preload on the connector may be taken into account.

In the initiation life method, the fatigue resistance of the connector is compared to the lowest fatigue resistance of the circumferential welds in the tubular members connected by the connector. The lowest circumferential weld resistance is characterized by the usual S–N curve. The fatigue resistance of the connector is determined in an equivalent form by determining the local stress ranges S' at notches in the connector, due to the same forces that cause S at the relevant circumferential weld in the tubular member. Pairs of S' versus the allowable number of cycles N' to crack initiation can be calculated using the local strain approach of Reference [A.15.2-2]. The effect of the mean stress at the notch due to make-up and preload on the connector should be included.

If the S'–N' curve characterizing the fatigue resistance of the connector is higher than the S–N curve characterizing the lowest fatigue resistance of the circumferential welds, the connector does not control the fatigue life of the connection. However, if the curves cross, the relative fatigue performance of the connector and the circumferential weld depends on the stress history. In this case, both fatigue lives should be calculated, using the same stress history as input for the fatigue life calculation of the connector and the fatigue life calculation of the circumferential weld.

A.15.2.6.2 Initiation life method

The calculation of initiation lives at notches in the connector requires

— detailed FEA of the connector to generate local stresses at the notches, and

— material data for both the monotonic (static) and the cyclic properties of the material in question.

Reference [A.15.2-2] contains one such set of data typical for connector material.

The crack initiation life or number of cycles to failure, N_f, at a generic notch of a mechanical connector may be evaluated by solving for the number of reversals to failure, $2N_f$, in the non-linear Equation (A.15.2-9):

$$\sigma_{re}\,\varepsilon_{re}\,E = \sigma'^{2}_{f}(2\,N_f)^{2b} + \sigma'_f\,\varepsilon'_f\,E(2\,N_f)^{b+c} \qquad \text{Solve for } N_f \tag{A.15.2-9}$$

where

σ_{re} is the equivalent fully-reversible elasto-plastic stress amplitude at the notch (not known);

ε_{re} is the equivalent fully-reversible elasto-plastic strain amplitude at the notch (not known);

E is the modulus of elasticity (material constant);

σ'_f is the fatigue strength coefficient (material constant);

ε'_f is the fatigue ductility coefficient (material constant);

b is the fatigue strength exponent (material constant);

c is the fatigue ductility exponent (material constant).

The coefficients and exponents defined above are generated experimentally for the material in question, and ε_{re} may be obtained using the cyclic stress-strain relation for the material using Equation (A.15.2-10).

$$\varepsilon_{re} = \frac{\sigma_{re}}{E} + \left(\frac{\sigma_{re}}{K'}\right)^{\frac{1}{n'}} \tag{A.15.2-10}$$

where

K', n' are the cyclic strain hardening coefficient and exponent, respectively, which are known as material constants and are determined experimentally;

σ_{re} is the equivalent fully reversible elasto-plastic stress amplitude at the notch (not known at this point).

The equivalent fully reversible stress amplitude at the notch, σ_{re}, includes the effect of the mean stress at the notch and may be calculated using Equation (A.15.2-11):

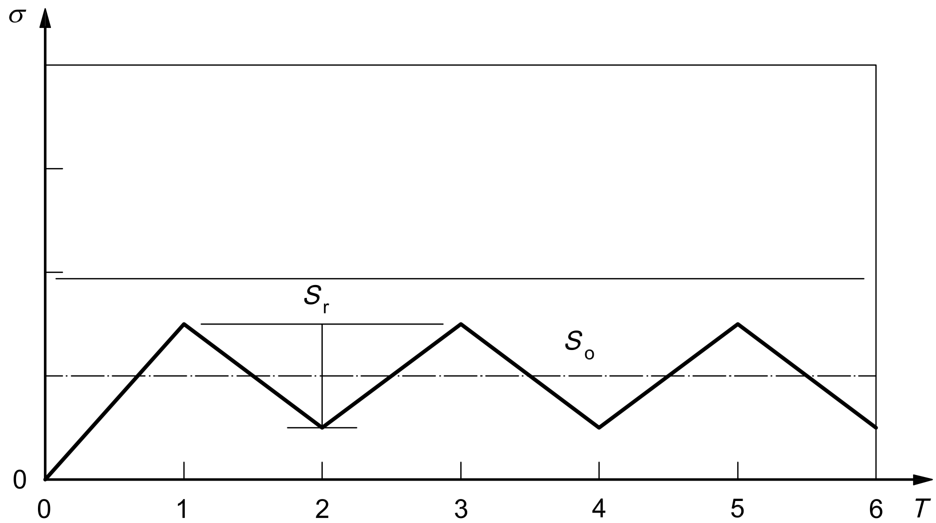
$$\sigma_{re} = \left[\frac{\sigma_a}{1 - \sigma_0 / \sigma_f'} \sqrt{(\sigma_0 + \sigma_a) \cdot \sigma_a} \right]^{0,5} \qquad \text{solve for } \sigma_{re} \qquad \text{(A.15.2-11)}$$

where

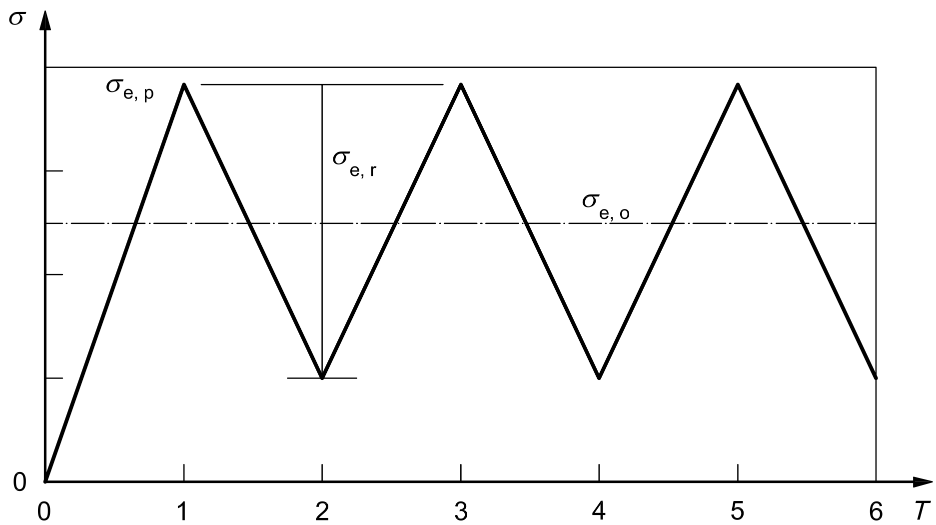
σ_0 is the elasto-plastic mean stress at the notch [see Figure A.15.2-2, c)] (not known at this point);

σ_a is the elasto-plastic stress amplitude at the notch, given by $\sigma_a = \frac{1}{2}\sigma_r$; and

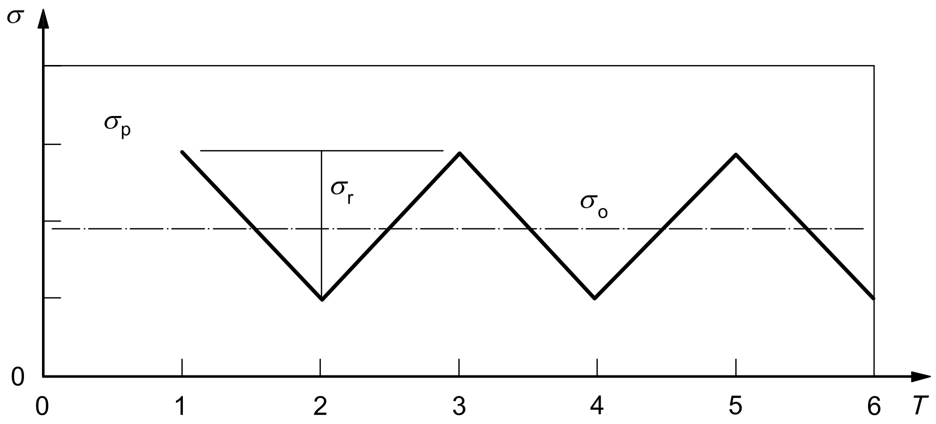
σ_r is the elasto-plastic stress range at the notch [see Figure A.15.2-2, c)] (not known at this point).



a) Nominal pipe stress variation



b) Elastic stress at notch (effect of SCF)



c) Transformed notch stress (effect of elasto-plastic behaviour)

Key

σ stress
T time step
S_r nominal pipe stress range
S_o mean nominal pipe stress
$\sigma_{e,p}$ peak elastic stress at notch
$\sigma_{e,r}$ elastic stress range at notch
$\sigma_{e,o}$ mean elastic stress at notch
σ_p peak elasto-plastic stress at notch
σ_r elasto-plastic stress range at notch
σ_o mean elasto-plastic stress at notch

Figure A.15.2-2 — Definition of pipe and notch stresses

The local elasto-plastic notch stresses, mean and range, may be obtained by transforming the elastic notch response obtained from FEA. This transformation accounts for the local plasticity that takes place at the notch. The elastic notch response may be given in terms of the stresses directly calculated at the notch, the nominal pipe tubular stresses and a geometric stress concentration factor (SCF), which relates the pipe tubular stress, S, to the notch stress, σ (preferred), or a combination thereof. The elastic notch stresses may be transformed to elasto-plastic stresses by one of two methods: the strain energy density or Neuber's.

a) Strain energy density method

The energy method simply equates the elastic strain energy at the notch to the elasto-plastic energy given by the actual cyclic stress-strain behaviour obtained experimentally for the material in question (see Figure A.15.2-3). This transformation is general and accurate. Knowing the elastic notch stress range and peak stress [Figure A.15.2-2, b)], Equations (A.15.2-12) to (A.15.2-14) apply:

$$\frac{\sigma_{e,r}^2}{2E} = \frac{\sigma_r^2}{2E} + \frac{\sigma_r}{n'+1}\left(\frac{\sigma_r}{K'}\right)^{1/n'} \quad \text{solve for } \sigma_r \tag{A.15.2-12}$$

and

$$\frac{\sigma_{e,p}^2}{2E} = \frac{\sigma_p^2}{2E} + \frac{\sigma_p}{n'+1}\left(\frac{\sigma_p}{K'}\right)^{1/n'} \quad \text{solve for } \sigma_p \tag{A.15.2-13}$$

where

$\sigma_{e,r}$ is the elastic stress range at the notch from FEA;

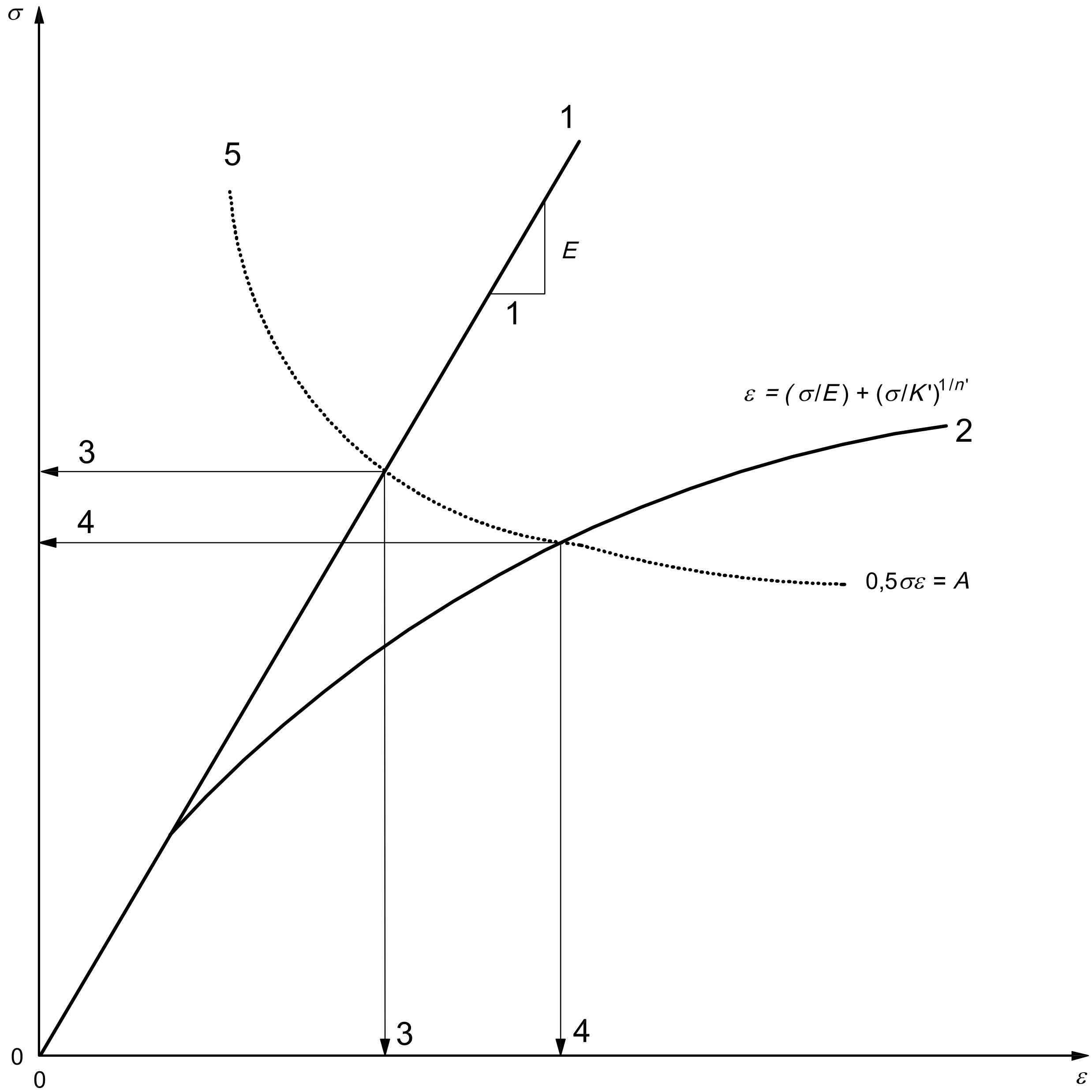
σ_r is the elasto-plastic stress range (transformed) at the notch;

$\sigma_{e,p}$ is the elastic peak stress at the notch from FEA;

σ_p is the elasto-plastic peak stress (transformed) at the notch.

Then, the elasto-plastic mean stress, σ_o, at the notch is

$$\sigma_o = \sigma_p - \frac{\sigma_r}{2} \tag{A.15.2-14}$$



Key

1 elastic behaviour

2 elasto-plastic behaviour

3 intersect with elastic behaviour

4 intersect with elasto-plastic behaviour

5 line of equal strain energy (area below stress-strain curve)

σ notch stress

ε notch strain

Figure A.15.2-3 — Transformation of elastic stress via strain energy

b) Neuber's method

Neuber's method represents an engineering approximation to the same transformation and is based on Neuber's rule (the geometric mean of the strain and stress concentration factors remains constant with load) and Peterson's empirical fatigue strength reduction factor, K_f, see Reference [A.15.2-3]. Referring to Figure A.15.2-4, the notch stress is linearly related to the pipe tubular stress via an SCF and there is no initial prestress at the notch. Two approaches can be followed to obtain the notch peak stress and stress range to calculate the notch mean stress.

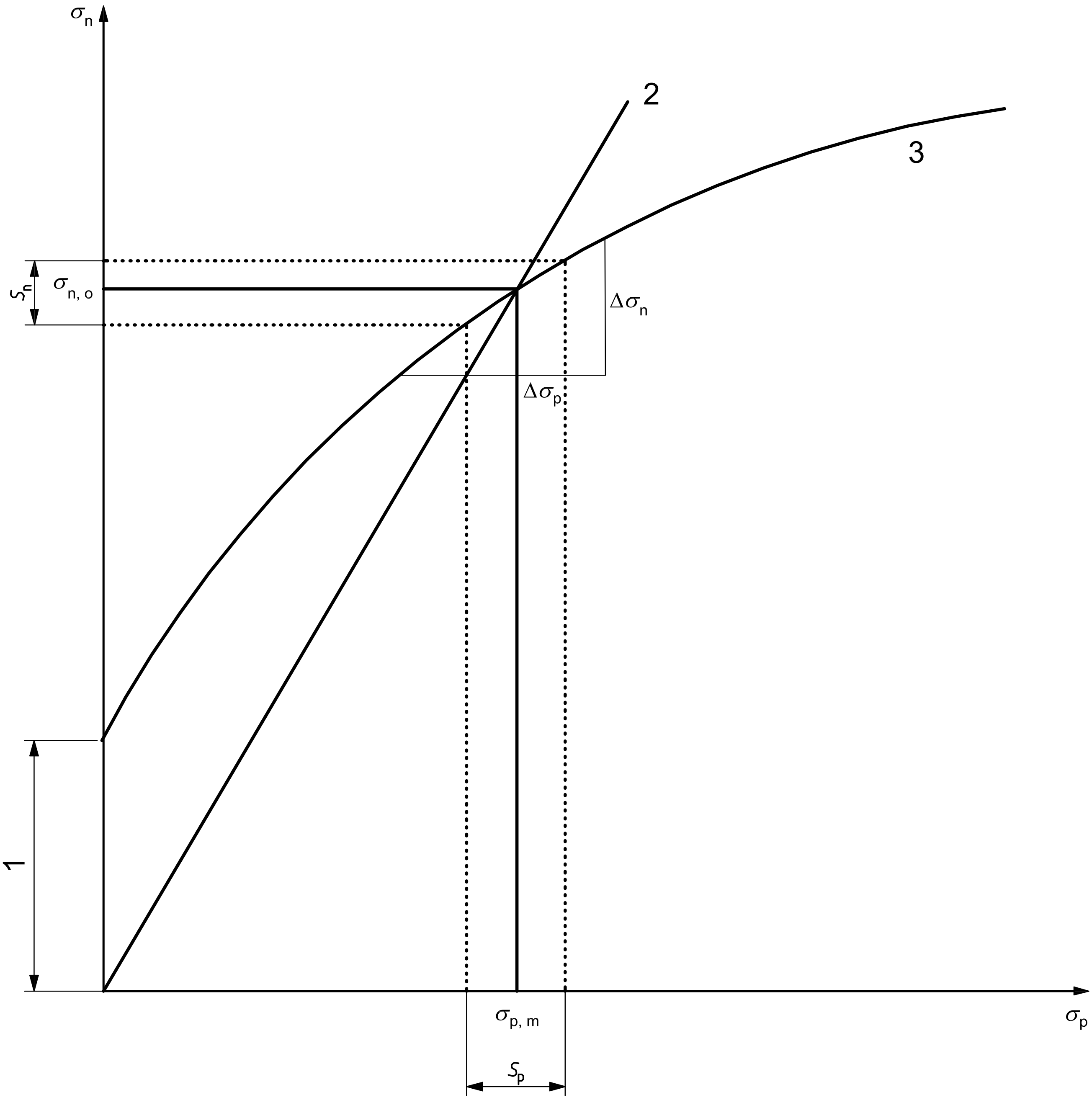


Figure A.15.2-4 — Relation between notch and pipe stresses

Knowing the nominal pipe tubular stress range (S_r), peak stress (S_o), the elastic SCF, the transformed elasto-plastic notch stress range, σ_r, and the peak notch stress, σ_p, can be obtained directly from the nominal pipe tubular stresses [Figure A.15.2-2, a)] according to the following two relations:

$$\sigma_r \left[\frac{\sigma_r}{E} + \left(\frac{\sigma_r}{K'} \right)^{1/n'} \right] = \frac{1}{E} \left(K_f \cdot S_r \right)^2 \qquad \text{solve for } \sigma_r \qquad \text{(A.15.2-15)}$$

and

$$\sigma_p \left[\frac{\sigma_p}{E} + \left(\frac{\sigma_p}{K'} \right)^{1/n'} \right] = \frac{1}{E} K_f^2 \left(S_o + \frac{S_r}{2} \right)^2 \qquad \text{solve for } \sigma_p \qquad \text{(A.15.2-16)}$$

where

S_r is the nominal stress range in the pipe tubular (known);

S_o is the mean nominal stress in the pipe tubular (known);

K_f is the fatigue strength reduction factor, or effective fatigue SCF given by

$$K_f = 1 + \frac{\text{SCF} - 1}{1 + A/r}$$

where

r is the Notch radius, in millimetres;

$\text{SCF} = \dfrac{\Delta\sigma}{\Delta S}$ known from FEA;

$A = 0{,}821 \left(\dfrac{300}{\sigma_u} \right)^{1{,}8}$;

σ_u is the ultimate stress of the material in megapascals.

Then, the elasto-plastic mean stress, σ_o, at the notch is

$$\sigma_o = \sigma_p - \frac{\sigma_r}{2} \qquad \text{(A.15.2-17)}$$

The accuracy of the transformation by Neuber’s method depends on the relative contribution of the connector preload to the notch stress and on the degree of non-linearity between the notch stress and the pipe tubular stress. If linearity between the notch and pipe tubular stresses exists and there is no preload, the notch stresses can be calculated using the pipe tubular stresses and an SCF, and the energy method can alternatively be used.

If the local notch stress is not linearly related to the pipe tubular stress and the connector is preloaded (Figure A.15.2-4), then

1) the SCF varies with the pipe tubular stress, and

2) there is an initial stress at the notch that is not related to the pipe tubular stress but to dimensional interferences in the connector.

In this case, the instantaneous SCF may be taken as the derivative of the notch stress with respect to the pipe tubular stress to relate the pipe tubular stress range to the notch stress range. However, the pipe tubular mean stress can no longer be related to the notch stress via an SCF. To avoid this situation, the elastic mean stress directly calculated at the notch via FEA is used.

Knowing the elastic mean notch stress (σ_o) and the pipe tubular stress range (S_r) and the instantaneous SCF, the transformed notch stress range, σ_r, and the notch peak stress, σ_p, can be obtained using Neuber's method by

$$\sigma_r \left[\frac{\sigma_r}{E} + \left(\frac{\sigma_r}{K'} \right)^{1/n'} \right] = \frac{1}{E} \left(K_f \cdot S_r \right)^2 \qquad \text{solve for } \sigma_r \qquad \text{(A.15.2-18)}$$

and

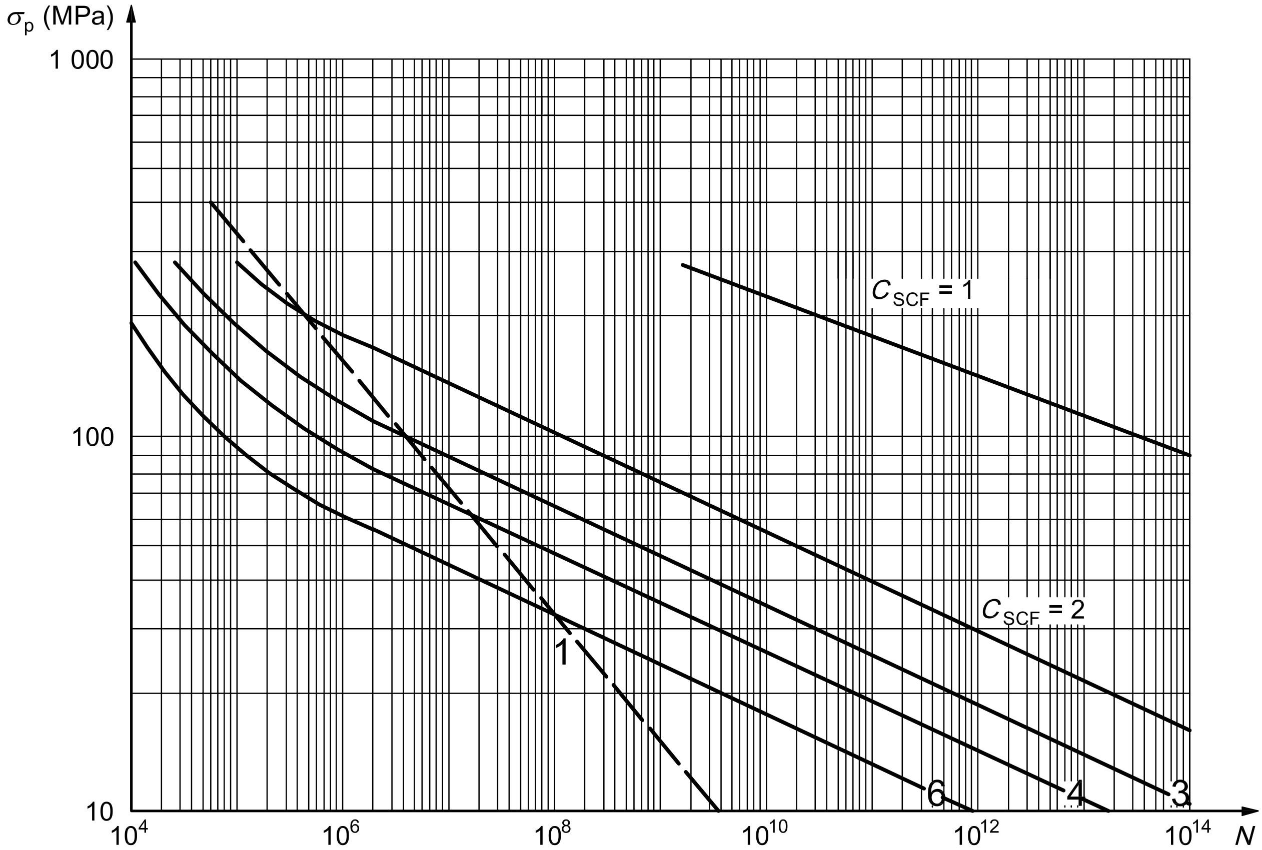
$$\sigma_p \left[\frac{\sigma_p}{E} + \left(\frac{\sigma_p}{K'} \right)^{1/n'} \right] = \frac{1}{E} \left[\sigma_{e,o} + K_f \left(\frac{S_r}{2} \right) \right]^2 \qquad \text{solve for } \sigma_p \qquad \text{(A.15.2-19)}$$

Again, the elasto-plastic mean stress, σ_o, at the notch is

$$\sigma_o = \sigma_p - \frac{\sigma_r}{2} \qquad \text{(A.15.2-20)}$$

Using σ_r, and σ_o in the equation for the equivalent, fully-reversible, σ_{re}, the corresponding strain, ε_{re}, can be calculated. With σ_{re} and ε_{re}, it is possible to solve for N_f in the first equation.

An example of S–N curves developed for an actual connector is presented in Figure A.15.2-5, compared to a ground flush circumferential weld estimated as the D curve from the UK HSE guidance, shifted by a factor of two on life. The connector in question develops preload upon makeup, the notch stress is not linear with the pipe tubular stress, and the nominal pipe tubular is maintained at a mean tension of 690 MPa. For this example, the last set of equations presented above was used to transform the stresses. The material used is HY-80 from Reference [A.15.2-2].



Key

1	*S–N* curve for ground circumferential weld
solid lines	*S–N* curves for varying SCF
σ_p	pipe stress
N	number of cycles to failure

HY-80 material property data

n	= 0,147
K'	= 1 184 MPa
σ_f'	= 1 249 MPa
ε_f'	= 1,456
a	= −0,101
b	= −0,693

Figure A.15.2-5 — Example of calculated *S–N* curves for various SCFs and constant pipe mean stress for HY-80 material compared with the circumferential weld between connector and pipe

A.15.2.7 Stress analysis validation

No guidance is offered.

A.15.2.8 Threaded fasteners

A.15.2.8.1 General

No guidance is offered.

A.15.2.8.2 Threaded fastener materials and manufacturing

Threaded fasteners should comply with the requirements of BS 4882[A.15.2-5] or similar.

Rolled threads, as opposed to cut threads, tend to improve fatigue performance, provided the rolling process is undertaken after the heat treatment. However, the S–N curve provided in A.15.2.8.7 does not take advantage of an improvement associated with rolled threads, if any. Furthermore, fatigue test results in air show only a marginal fatigue performance improvement due to rolling of the threads [A.15.2-6].

If threaded fasteners or studs are partially threaded, the location and geometry of thread run-out should be such that stress concentrations are minimized.

A.15.2.8.3 Threaded fastener installation

Threaded fasteners may be preloaded by controlling the applied load, the stretch or the torque. The load control method to preload threaded fasteners is considered the most reliable. In this case, threaded fastener preload can be achieved by applying an axial load to the threaded fastener and running the nut to a hand-tight position before releasing the load. Typically, hydraulic bolt tensioners are used, which are modified hydraulic jacks. When allowed to react against a rigid, even support and calibrated to a standard traceable to a recognized national standard, axial hydraulic bolt tensioners can provide a very precise prestress.

When the load is released from the tensioner and transferred to the nut, the resulting deformations as the nut engages the threaded fastener lead to what is known as a transfer loss. This loss should be taken into account during tensioning. The percentage of transfer losses for standard metric and UNS (unified screw thread, standard series) threads bearing on a rigid steel plate may be taken as

$$\Delta P_{\text{ave}} = \left(0{,}9\frac{d}{l}\right)\cdot 100\ \% \qquad \text{(A.15.2-21)}$$

$$\Delta P_{\text{max}} = \left(1{,}0\frac{d}{l}\right)\cdot 100\ \% \qquad \text{(A.15.2-22)}$$

where

ΔP_{ave} is the average percentage transfer loss;

ΔP_{max} is the maximum percentage transfer loss;

d is the threaded fastener diameter;

l is the stressed length of the threaded fastener, which can be taken as the distance between the first and the last engaged threads.

Where the surface supporting fasteners is flexible, reduced transfer losses will occur. The reduction in losses may be taken into account by calculating an equivalent effective stressed length.

When using hydraulic tensioners, care should be taken to ensure that threaded fasteners are not tensioned beyond yield, since this may lead to excessive deformation. The maximum tensioner load applied to the threaded fastener should not exceed the yield load less a safety margin equal to two standard deviations, as obtained from the tensioning calibration test.

In the absence of specific data, the threaded fastener preload should also include a material relaxation loss of approximately 2 % for steel threaded fasteners with a minimum yield strength above 600 MPa at room temperature. Steel threaded fasteners with a minimum yield strength less than 600 MPa are not recommended for use where a controlled preload is required.

Other methods for preloading threaded fasteners are direct torquing and the turn of the nut, a stretch controlled method. The latter method differs from the former in that a specified torque is initially applied, followed by a measured rotation between the nut and the threaded fastener. Although more practical than direct preload, these methods are inherently less reliable due to the friction on the threads and between the nut and the bearing surface. Use of the direct torque method is not recommended unless carefully calibrated tests, traceable to a national standard, are carried out in order to consistently account for friction. Control of lubrication and avoidance of galling is important to achieve consistent results.

For threaded fastener connections intended for multiple assembly and disassembly operations, in which the threaded fasteners are to be reused, the calibration procedure should include multiple make-up and break-out operations.

A.15.2.8.4 Threaded fastener inspection

The level of cathodic protection of individual bolts should be checked. The cathodic potential of any individual bolt should be minus 900 mV ± 50 mV compared with an Ag-AgCl cell.

When inspecting the bolts for tension, the bolts should be re-tensioned to the same level to which they were intended to be preloaded during installation, using the original installation procedure and calibrated tools. If any bolt is found to fall below the minimum design value, the bolt should be re-tensioned to its originally required value.

A.15.2.8.5 Threaded fastener strength criteria

The extreme fibre stress in the connector (axial plus bending), induced by preload and factored actions on the connected tubular members, should satisfy the following criteria

$$\sigma_{\mathrm{bt}} \leqslant \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{R,bt}}} \qquad \text{(A.15.2-23)}$$

where

σ_{bt} is the extreme fibre stress (axial plus bending), acting on the minimum section;

f_{y} is the representative yield strength of the material;

$\gamma_{\mathrm{R,bt}}$ is the resistance factor for axial tensile strength = 1,0.

In addition, the size and number of threaded tension fasteners used should be such that the total factored tension force, P_{t}, due to factored actions, acting on the connected tubular members, does not exceed

$$P_{\mathrm{t}} \leqslant \frac{f_{\mathrm{y}}\ A_{\mathrm{b}}\ N_{\mathrm{b}}}{\gamma_{\mathrm{R,ba}}} \qquad \text{(A.15.2-24)}$$

where, additionally,

A_{b} is the area of the shank of the threaded element;

N_{b} is the total number of threaded elements;

$\gamma_{\mathrm{R,ba}}$ is the resistance factor for threaded fastener area = 1,15.

A.15.2.8.6

Allowance should be made for time-dependent relaxation losses in threaded fasteners, the magnitudes of which depend on the material and service temperature. In determining the pre-tensioning force for a threaded fastener, allowance should also be made for losses associated with the pre-tensioning procedure, such as transfer losses, as well as for the precision of the procedure itself.

A.15.2.8.7 Threaded fasteners fatigue criteria

To minimize the stress fluctuation response in threaded fasteners caused by fluctuating actions, the fastener stressed length and cross-sectional area should be chosen to give maximum flexibility of the fastener within the constraints of the design (i.e. maximize the fastener preload extension).

FEA may be used to evaluate threaded fastener stress variations due to fluctuating actions. The FEA model for this case may be developed with the intent of only simulating stiffness, requiring only a coarse mesh. Ignoring the fastener preload in evaluating the fastener force variations is always conservative.

To calculate fatigue damage, Palmgren-Miner's sum may be used, in which the damages induced in the fastener by each of the stress variations are linearly superimposed. The damage due to a given variable force may be expressed in closed form when a spectral fatigue analysis of the structure has been performed. Alternatively, The damage may be calculated as the discrete sum of the ratios of the acting number of cycles of a given stress range in the fastener, as defined by the long-term distribution of annual occurrence versus stress range, to the allowed number of cycles for the same stress range, as given by an S–N curve.

In the absence of specific fatigue data for threaded fasteners, the following S–N curve may be used, however the user should be aware that the derivation of Equation (A.15.2-25) does not necessarily account for the effects of preload:

$$\log N = 11{,}55 - 3 \log S \qquad \text{(A.15.2-25)}$$

where N is the number of cycles to failure and S is the stress range in the fastener.

The S–N curve provided above is lower bound to fatigue tests in air given in Reference [A.15.2-6] augmented by tests conducted at TWI in the UK on behalf of HSE [A.15.2-7]. The data include tests in which the applied load is transferred to the fastener from the loading plate via a single or double nut arrangement. In the latter arrangement, the load is transferred from a perforated plate, through which the fastener passes. The fastener is connected to the plate by means of two nuts torqued against each other, one on each side of the plate. The above S–N curve provides for reasonably conservative designs for fasteners made of ASTM A193 [A.15.2-8], Grade B7 and L7 steels, as well as of ISO Grades 8,8 and 12,2 to BS 3692 [A.15.2-9].

Unless demonstrated by pertinent tests under relevant conditions, no advantage may be taken of an endurance limit.

Although fatigue data show that rolling the threads after heat treatment tends to improve the fatigue performance of fasteners, the above given S–N curve conservatively ignores this effect. Heat treating fasteners after rolling the threads essentially results in a fastener with a fatigue performance similar to that of fasteners with cut threads.

Tests conducted at TWI in the UK showed that the fatigue performance of Grades 8.8, B7 and L7 bolts, tested under cathodically protected conditions, experienced reductions on fatigue life relative to in-air performance. Therefore, a reduction factor of 3,0 on the life obtained by the S–N curve given above may be used to account for the effect of using bolts underwater with adequate cathodic protection in the range –850 mV to –1 000 mV. However, the underwater fatigue performance of fasteners made of higher grade steels should be specifically addressed via a validated test programme, as should the effect of hydrogen embrittlement.

Tests on threaded fasteners varying from 19 mm to 44 mm in diameter made of Grades 8.8, B7 and L7 support a correction factor on stress, relative to the 20 mm size, given by Equation (A.15.2-26):

$$k_{\text{corr}} = \left(\frac{d}{20} \right)^{0,3} \qquad \text{(A.15.2-26)}$$

where d is the fastener diameter, in millimetres.

The factor, k_{corr}, is applied to the stress range prior to entering the S–N curve given above.

A.15.2.9 Swaged connections

No guidance is offered.

A.15.3 Clamps for strengthening and repair

A.15.3.1 General

The general clamp design process is presented in Figure 15.3-1.

A.15.3.2 Split-sleeve clamps

No guidance is offered.

A.15.3.3 Prestressed clamps

No guidance is offered.

A.15.3.4 Forces on clamps

A.15.3.4.1 Mechanism of force transfer

No guidance is offered.

A.15.3.4.2 Member forces

No guidance is offered.

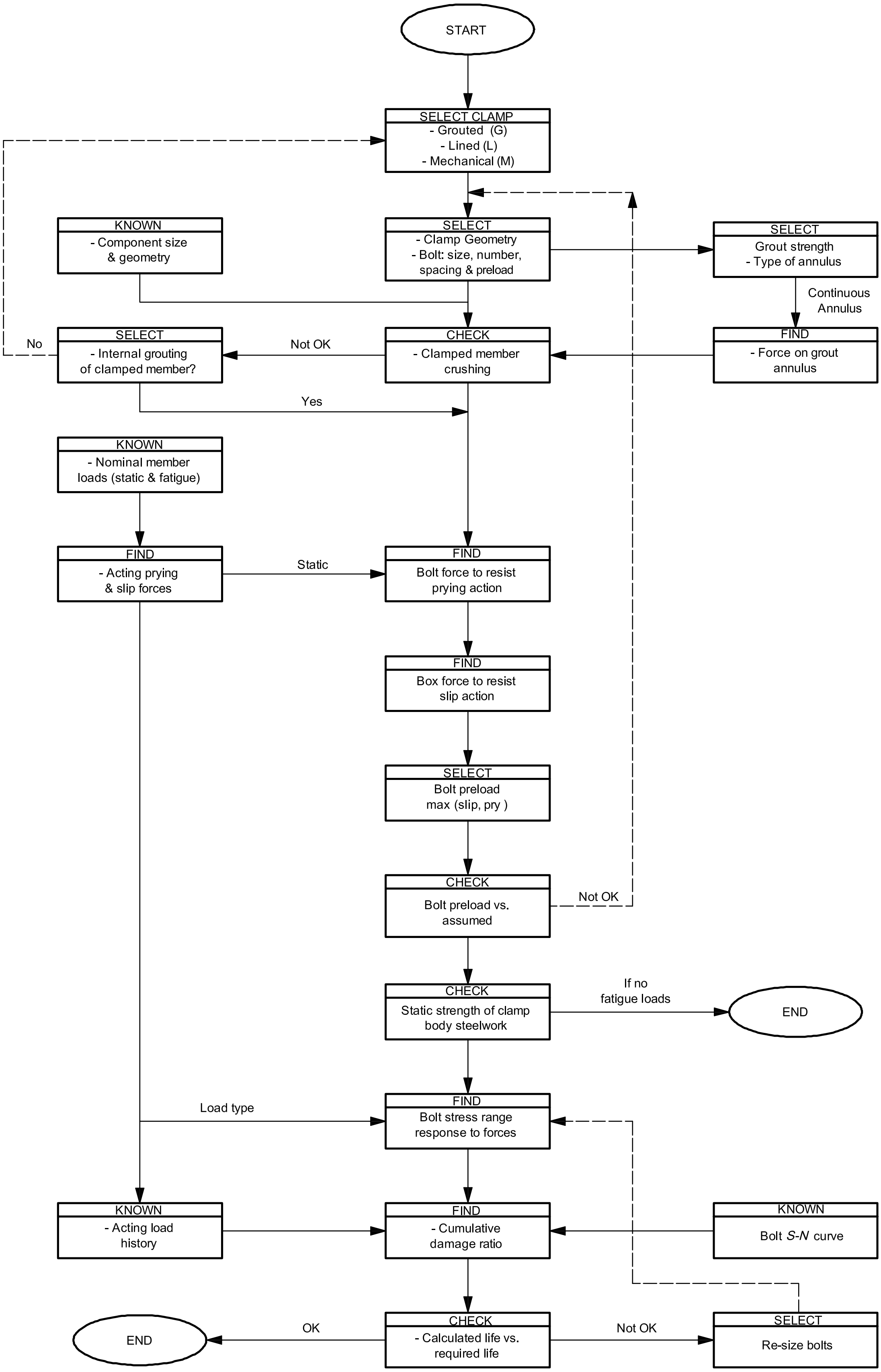


Figure A.15.3-1 — General design procedure for stressed clamps

A.15.3.4.3 Bolt forces

A.15.3.4.3.1 General

Figures A.15.3-2, A.15.3-3 and A.15.3-4 illustrate typical applications of clamps for strengthening or repair of structures for, respectively:

a) end-to-end connections;

b) addition of members, and

c) tubular joints.

The capability of prestressed clamps to transfer forces is determined by the bolt prestress. Bolts should be prestressed so that the clamp does not open or slip due to the forces acting on the clamp. Shear forces and bending moments from the substrate member tend to pry or separate the clamp, whereas axial force and torsion tend to cause the clamp to slip.

The prying action on the clamp arising from the shear forces and bending moments is directly resisted by extension in the bolts. However, the forces that tend to pull out or slip the substrate member from the clamp are resisted by the friction developed at the clamp-member interface. This frictional resistance is induced by the contact pressure developed as the bolts are tightened or prestressed. Because the prying action tends to relieve the contact pressure and, thus, decrease the resistance against slippage, the bolt prestress should correspond to the greater of the following two bolt forces:

— bolt forces required to resist the prying (without separation);

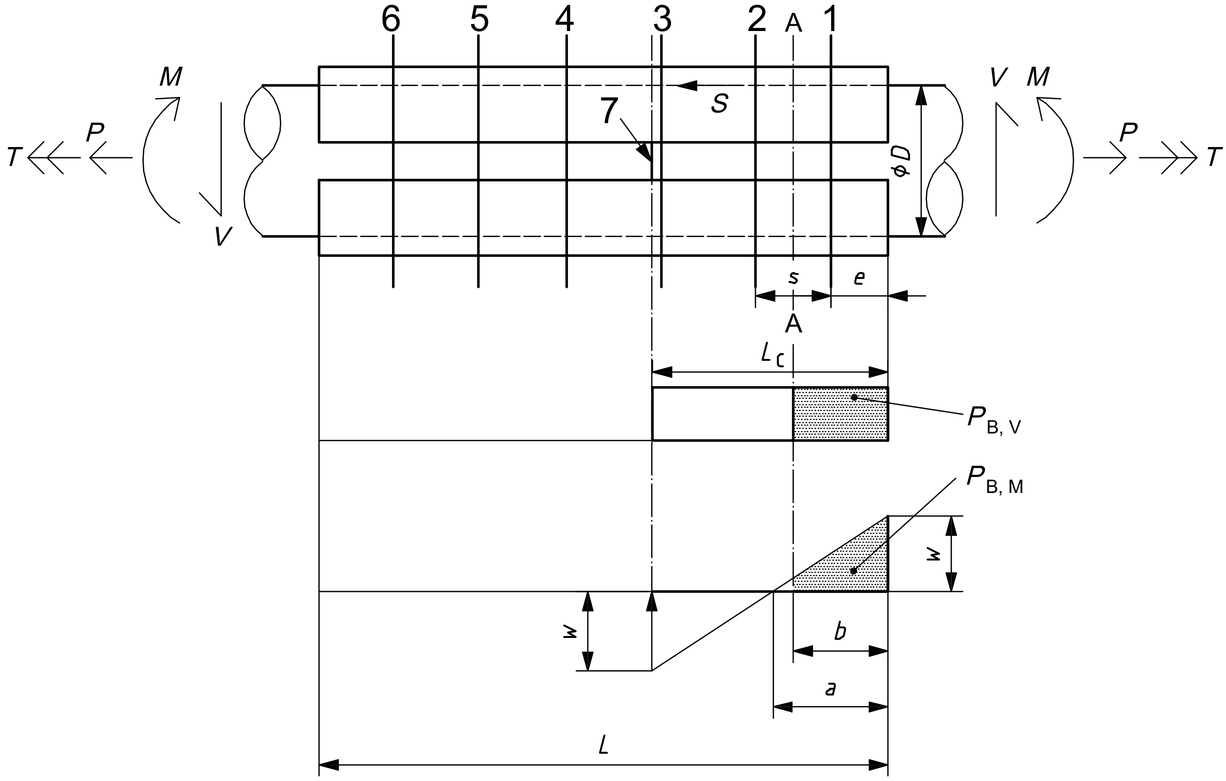
— bolt forces required to prevent slippage.

When calculating the bolt forces in clamps intended to strengthen or repair joints between tubulars, advantage may be taken of the sharing of forces between the clamp and the substrate member, provided that such load sharing of forces is demonstrated by analysis or tests. Otherwise, it is conservative to assume that no sharing of forces takes place and that the clamp fully transfers the substrate forces across the joint.

Methods for calculating the required prestressing forces in the bolts to prevent prying and slipping at the clamp-member interface for each of the three clamp applications are presented in A.15.3.4.3.2 to A.15.3.4.3.4, ignoring sharing of forces between the clamp and the substrate member.

A.15.3.4.3.2 End-to-end connection clamps

Figure A.15.3-2 shows a generic clamp over an end-to-end connection and a typical set of substrate member end forces. The prying forces from the substrate member are transferred to the clamp and, thus, to the bolts over the contact length L_c. To calculate the axial force sustained by the bolts due to the prying forces, in the absence of test data, it may conservatively be assumed that the vectorial resultants from the acting prying moments and shear forces superimpose on one another to separate the clamp (Figure A.15.3-3, section A-A).



Key

1-6	bolts
7	end of substrate member on the right hand side
D	member diameter
L	length of clamp
L_c	contact length on substrate member
$P_{B,V}$	total shear force on end bolt due to shear force V on substrate member (shaded area)
$P_{B,M}$	total force on end bolt due to bending moment M on substrate member (shaded area)
w	maximum force per unit length due to bending moment M
S	slip force

Figure A.15.3-2 — Typical application of a clamp on an end-to end connection

The resultants of the in-plane and out-of-plane bending moments and shears can be taken as

$$M = \sqrt{\left(M_{\text{ipb}}\right)^2 + \left(M_{\text{opb}}\right)^2} \tag{A.15.3-1}$$

and

$$V = \sqrt{\left(V_{\text{ip}}\right)^2 + \left(V_{\text{op}}\right)^2} \tag{A.15.3-2}$$

where

M is the resultant bending moment on the substrate member assumed acting as if in-plane;

M_{ipb} is the in-plane bending moment on the substrate member;

M_{opb} is the out-of-plane bending moment on the substrate member;

V is the resultant shear force on the substrate member assumed acting as if in-plane;

V_{ip} is the in-plane shear force on the substrate member;

V_{op} is the out-of-plane shear force on the substrate member.

Tests of an end-to-end joint have shown that in the case of grouted clamps with the bolts oriented as shown in Figure A.15.3-2, only a fraction of the applied out-of-plane bending moment (M_{opb}) results in bolt force variation so long as no separation of the clamp halves occurs. In absence of pertinent data, one half of the acting M_{opb} can be substituted in the above equation when calculating the total bending moment, M. However, for neoprene-lined clamps, tests show that the full M_{opb} should be used.

The overall separation action on the clamp results in a contact pressure distribution at the clamp-member interface that is maximum at the free end of the clamp and decreases toward the centre of the clamp. The distribution of this pressure and the length over which it acts is not well understood. In the absence of a detailed analytical or experimental evaluation of such pressure, the pressure distribution arising from the resulting bending moment (M) may be assumed to decrease over a distance, $2a$, from the free end, where it is maximum, as indicated in Figure A.15.3-2. The distance $2a$ can be taken as equal to the contact length, L_c. The pressure arising from the resulting shear force (V) may be assumed to be uniform over the same length ($2a$) over which the bending moment was assumed to have dissipated.

In general, the force resisted by the end bolts due to prying action should include the effect of the resultant bending moment and resultant shear force and, if applicable, any other components of forces acting directly on the clamp (e.g. add-on members) that are parallel to the bolt axes. Thus, the bolt force, $P_{B,b}$, assuming two rows of bolts parallel with the axis of the substrate members (one on either side) as shown in Figure A.15.3-2, can in general be given by

$$P_{B,b} = \tfrac{1}{2}\left(P_{B,M} + P_{B,V} + P_{B,P}\right) \tag{A.15.3-3}$$

where

$P_{B,b}$ is the total force on the end bolt (assuming two parallel rows of bolts);

$P_{B,M}$ is the force on the end bolt induced by the resultant bending moment, M;

$P_{B,V}$ is the force on the end bolt induced by the resultant shear force, V;

$P_{B,P}$ is the bolt force induced by other forces parallel to the bolt axes and acting on the clamp, if present.

For clamps installed over end-to-end connections $P_{B,P} = 0$, therefore, only the bending moments and shear forces on the substrate member contribute to the bolt force through $P_{B,M}$ and $P_{B,V}$. As indicated in Figure A.15.3-2 by the shaded area, these forces can generally be obtained from the corresponding pressure distributions brought about by the bending moment and shear force resultants acting over a distance equal to one half of the bolt spacing on each side of the bolt.

The force on the end bolt induced by the resultant shear force is equal to the average shear force over the contact length ($V/2a$) times the distance, b:

$$P_{B,V} = \frac{V}{2a} \cdot b \tag{A.15.3-4}$$

where, further to previous definitions,

a $= L_c/2$;

b is the larger of the two distances s and ($s/2 + e$);

s is the bolt spacing;

e is the distance between the end bolt and the end of the clamp (see Figure A.15.3-2).

In addition, the substrate member tends to slip relative to the clamp. The total slip force may be taken as the vectorial sum of the slip forces in the longitudinal and circumferential directions. The slip force associated with the longitudinal shear force induced by the bending moment (M) in the substrate member may be ignored. Hence, the total acting slip force is according to Equation (A.15.3-5):

$$P_{\mathrm{Sl}} = \sqrt{P_{\mathrm{ax}}^2 + \left(\frac{2T}{D}\right)^2} \tag{A.15.3-5}$$

where

P_{Sl} is the slip force between the substrate member and the clamp;

P_{ax} is the axial force on the substrate member;

T is the torsional moment on the substrate member;

D is the outside diameter of the substrate member (see Figure A.15.3-2).

The minimum bolt force required to prevent slip of the clamp can be obtained by comparing the acting slip or interface transfer stress to the corresponding strength as given in A.15.3.5.5.

The interface transfer stress σ_{p} acting on the clamp is according to Equation (A.15.3-6):

$$\sigma_{\mathrm{p}} = \frac{S}{\pi D L_{\mathrm{s}}} \tag{A.15.3-6}$$

where

S is the slip force;

L_{s} is the length of the clamp over which the slip force is assumed to transfer.

For clamps used in end-to-end joints, L_{s} should be taken as the shortest length from the joint to either end of the clamp, i.e. the shortest contact length L_{c}, if the clamp is not placed symmetrically relative to the joint. Both halves of the clamp are assumed to contribute to the slip resistance.

A.15.3.4.3.3 Addition-of-member clamps

Figure A.15.3-3 illustrates the use of clamps to add a member. In this instance, the substrate member only supports the clamp and the forces acting on the add-on member tend to pry and slide the clamp relative to the substrate member.

The prying moment,M, may be taken as

$$M = \sqrt{\left(M_{\mathrm{opb}} \cos\theta + T \sin\theta\right)^2 + \left(M_{\mathrm{ipb}}\right)^2} \tag{A.15.3-7}$$

where

M is the total prying moment on the clamp;

M_{ipb} is the in-plane bending moment on the added member;

M_{opb} is the out-of-plane bending moment on the added member;

T is the torsional moment on the added member;

θ is the included angle between the added member and the substrate member (see Figure A.15.3-3).

Both M_{ipb} and M_{opb} are calculated about the intersection of the add-on member axis and the axis of the substrate member (the work point).

The bolt force, $P_{B,b}$, is analogous to Equation (A.15.3-3), without a contribution from shear forces in the substrate member:

$$P_{B,b} = \frac{1}{2}\left(P_{B,M} + P_{B,P}\right) \tag{A.15.3-8}$$

where

$P_{B,b}$ is the total force on the end bolt (assuming two parallel rows of bolts);

$P_{B,M}$ is the force on the end bolt pair induced by the total prying moment, M; $P_{B,M}$ is calculated as for end-to-end clamps (see A.15.3.4.3.2);

$P_{B,P}$ is the force on the end bolt pair parallel to the bolt axes, induced by the forces in the added member

$$P_{B,P} = \frac{P \sin\theta - V \cos\theta}{N_{bp}} \tag{A.15.3-9}$$

where

P is the axial force on the added brace;

V is the resultant shear force on the added brace;

N_{bp} is the number of bolt pairs.

The total acting slip force can be taken as

$$P_{Sl} = \sqrt{\left(P \cos\theta + V_{ip} \sin\theta\right)^2 + \left(\frac{2\left(T \cos\theta + M_{opb} \sin\theta\right)}{D} + V_{op}\right)^2} \tag{A.15.3-10}$$

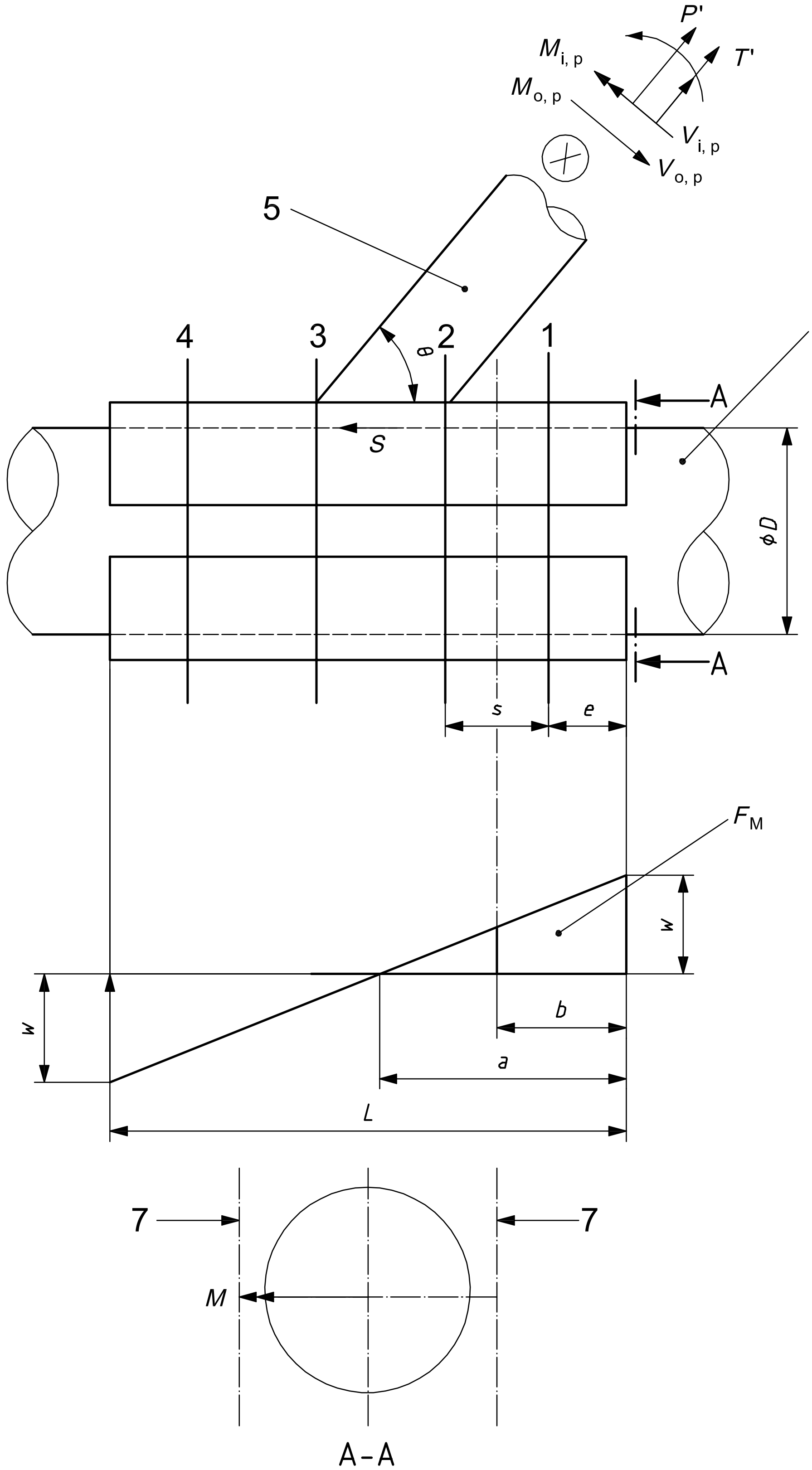
where, further to previous definitions,

P_{Sl} is the slip force between the substrate member and the clamp;

V_{ip} is the in-plane shear force on the added member;

V_{op} is the out-of-plane shear force on the added member;

D is the outside diameter of the substrate member.



Key

1-4 bolts

5 add-on member

6 substrate member

D diameter of substrate member

S slip force

s bolt spacing

w unit force due to acting prying moment M

M prying moment [Equation (A.15.3-8)]

Figure A.15.3-3 — Typical application of a clamp for the addition of a member

The minimum bolt prestress force required to prevent slip of the clamp can be obtained by comparing the acting slip or interface transfer stress to the corresponding strength as given in A.15.3.5.5.

The interface transfer stress is assumed resisted by that half of the clamp to which the add-on member is attached, and can be determined as

$$\sigma_p = \frac{1}{2} \cdot \frac{S}{\pi D L_s} f_p = \frac{1}{2} \cdot \frac{S}{\pi D L_s} \qquad \text{(A.15.3-11)}$$

where, furthermore,

σ_p is the interface transfer stress;

L_s is the length of the clamp over which the slip force is assumed to transfer.

The above slip force is assumed to act only on the clamp's half to which the add-on member attaches. The other half of the clamp is not active due to the inability of the bolts to transfer shear. However, both halves of the clamp can be made to act simultaneously by providing shear transfer between the two clamp halves using rigidly engaged longitudinal hinges, placed on each side of the clamp.

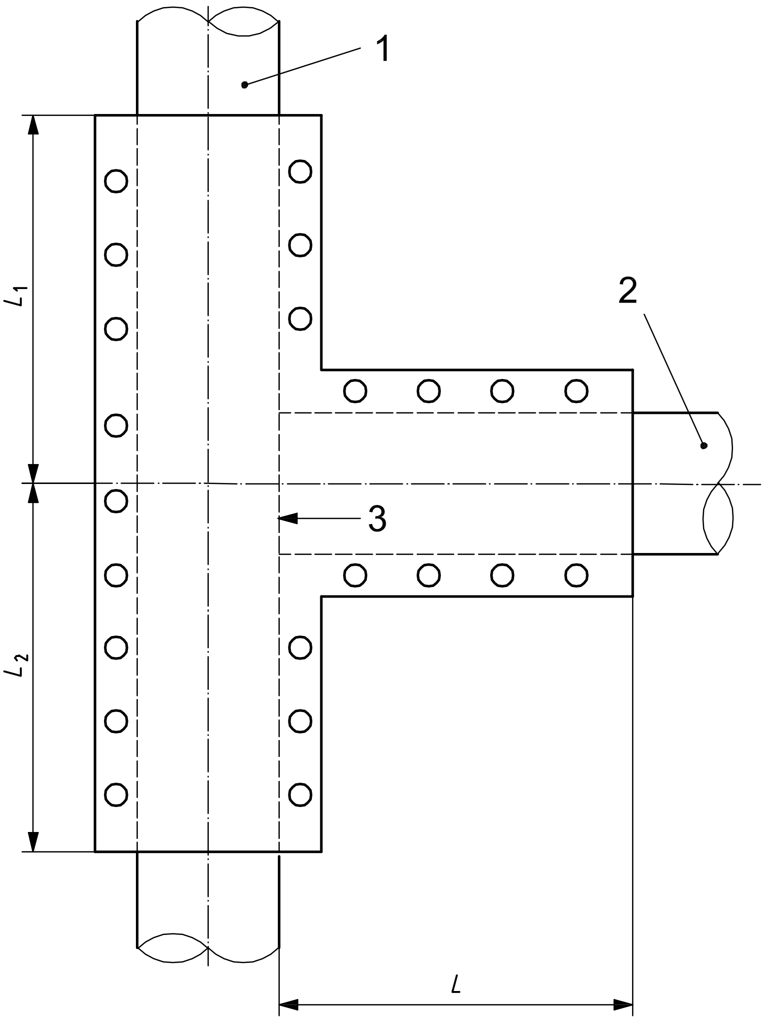
A.15.3.4.3.4 Clamps on tubular joints

Referring to Figure A.15.3-4, the prying forces on clamps placed on tubular joints may conservatively be determined by treating each of the tubular joint member ends as the substrate member of a clamp on an end-to-end connection. The substrate member in Figure A.15.3-4 can be the brace or the chord parts of the tubular joint. Assuming that the clamp is to fully transfer the forces, the lengths over which force transfer can be assumed to take place are

a) L for the brace side, and

b) L_1 and L_2 for the chord sides.

Likewise, the calculation of the slip force may conservatively be treated as a case of an end-to-end clamp for each of the free ends of the joint.



Key

1 chord substrate member

2 brace substrate member

3 break

Figure A.15.3-4 — Typical application of a clamp on a tubular joint

A.15.3.5 Clamp design

A.15.3.5.1 General approach

No guidance is offered.

A.15.3.5.2 Check of the clamped member

No guidance is offered.

A.15.3.5.3 Static design of bolts

No guidance is offered.

A.15.3.5.4 Fatigue design of bolts

Finite element analysis can be used to evaluate bolt force fluctuations due to the time-varying actions. The FEA meshes in analyses used for evaluating force fluctuations may be coarse and the geometry may be defined by mid-surface planes. The bolts may be modelled with single beam elements, and the clamp and the substrate members may be modelled with plate and thin shell elements. Alternatives for modelling the bolt preload and clamp-member interface include

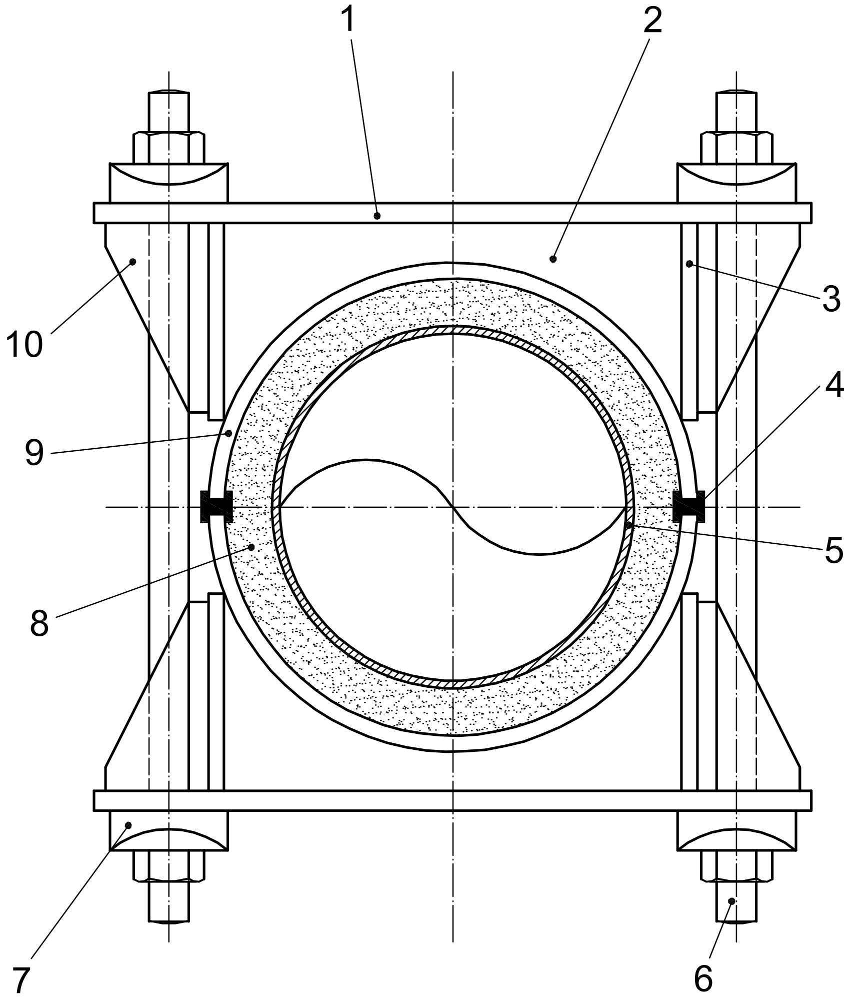
- explicitly modelling a bolt preload which is sufficient to maintain contact between the clamp and the substrate member, or
- ignoring the bolt preload and modelling the interface between the clamp and the substrate member with contact elements which allow gaps to open.

In FEA, the bolts should be prestressed to the intended level prior to applying any member forces to the clamp. Then, a linear elastic analysis should be conducted for each of the member forces relevant to the clamp in question and the corresponding variation of the axial stresses in the bolt determined. In this way, transfer functions between the forces on the clamp and the bolt stresses can be established. Hence, knowing the clamp force histories and the corresponding transfer functions, the actual stress history in the bolt can be obtained.

A.15.3.5.5 Interface transfer strength of prestressed clamps

The slip strength equations for prestressed clamps given below are based on a relatively limited experimental database [A.15.3-1], primarily with respect to the clamp configuration and distribution of bolts. In using this equation, designers should be aware of the limitations of the database. The single most important characteristic of the clamp cross-section is that it has a continuous top plate and is very rigid compared to the substrate member (see Figure A.15.3-5).

The database corresponds to pull tests on a single clamp geometry having a cross-section as shown in Figure A.15.3-2 (end-to-end connection). Three bolt sizes — M20, M32, and M40 — of constant length and at a constant spacing and prestress are included in the database. The clamp length-to-member diameter ratio varies from 0,5 to 2,0 and the member diameter-to-thickness ratios from 20 to 50. The steel surfaces were shot blasted, and the grout strength was greater than 50 MPa at 28 days for all tests.



Key

1 top plate
2 end plate
3 side plate
4 longitudinal seal
5 substrate member
6 long studbolts
7 spherical washer set
8 grout annulus
9 saddle
10 gussets

Figure A.15.3-5 — Typical clamp cross-section

For prestressed grouted clamps, the acting interface transfer stress (σ_p) due to factored actions should satisfy the following condition:

$$\sigma_p \leqslant \frac{f_g}{\gamma_{R,g}} \quad \text{(A.15.3-12)}$$

where

σ_p is the interface transfer stress at the substrate member;

$\gamma_{R,g}$ is the partial resistance factor for interface transfer strength for prestressed clamps = 2,0;

f_g is the representative interface transfer strength, in stress units, given by

$$f_g = 2C_p\ K_m^{0,6}\ f_{cu}^{0,3} + \rho_c \left(\frac{P_{B,n}}{\frac{\pi}{2} D_p\ L_s} \right) \tag{A.15.3-13}$$

where

C_p is the scale factor for the diameter of the substrate member as defined in 15.1.5.1;

K_m is the (modified) radial stiffness factor (see below);

f_{cu} is the specified unconfined strength of the grout as defined in 15.1.5.1;

ρ_c is a representative friction factor at the grout-steel interface (see below);

$P_{B,n}$ is the total bolt force

$$P_{B,n} = nP_{B,b}$$

where n is the number of bolts and $P_{B,b}$ is the prestressing force in one bolt;

D_p is the outside diameter of the substrate member;

L_s is the active length of the clamp.

The radial stiffness factor, K_m, is a modification of that used in grouted connections (see 15.1.5.1) to account for the reduction in hoop stiffness due to the presence of the bolts, and is given Equation (A.15.3-14):

$$K_m = \left(\frac{D_p}{t_p} + \frac{s_n L_n}{A_n} \right)^{-1} + \frac{1}{m} \left(\frac{D_g}{t_g} \right)^{-1} \tag{A.15.3-14}$$

where

D_p is the outside diameter of the substrate member;

t_p is the wall thickness of the substrate member;

s_n is the bolt spacing;

L_n is the stressed length of the bolt;

A_n is the cross-sectional area of the bolt;

m is the ratio of elastic moduli of steel and grout $= E_s/E_g$;

D_g is the outside diameter of the grout annulus;

t_g is the thickness of the grout annulus.

The representative friction factor at the grout-steel interface (not the classical coefficient of friction) can be determined from

$$\rho_c = 0{,}15\left(1 + 30K_m^{0,6}\right) \tag{A.15.3-15}$$

The representative interface transfer strength Equation (A.15.3-13) includes the contributions from

a) the plain member grouted connection as given in 15.1.6 for $h/s = 0$, but with a modified radial stiffness factor, K_m, and

b) the friction developed at the grout-member interface due to the prestress in the bolts.

For prestressed mechanical clamps, the acting interface transfer stress, σ_p, due to factored actions should also satisfy the condition:

$$\sigma_p \leqslant \frac{f_g}{\gamma_{R,g}} \tag{A.15.3-16}$$

with the representative interface transfer strength given by

$$f_g = \rho_c \left(\frac{P_{B,n}}{\pi/2\, D_p L_s} \right) \tag{A.15.3-17}$$

with

$$\rho_c = 0{,}13\ (1 + 30\ K_m^{0{,}6}) \tag{A.15.3-18}$$

$$K_m = \left(\frac{D_p}{t_p} + \frac{s_n L_n}{A_n} \right)^{-1} \tag{A.15.3-19}$$

In this case, only the steel to steel friction contributes to the interface transfer strength of the clamp, which is in turn a function of the substrate member and bolt stiffnesses.

As in the case of grouted clamps, the transfer strength equation for mechanical clamps is based on limited data. The restrictions imposed on the database for the grouted clamps also apply to the mechanical clamps, with the caveat that mechanical clamps are much more sensitive to fit-up than grouted clamps are. Hence, in designing mechanical clamps, the obtaining of the actual dimensions of the structure upon which the clamp is to be installed is strongly recommended.

For prestressed lined clamps, representative interface strength values should be determined by tests. In the absence of any such test data, the acting interface transfer stress, σ_p, due to factored actions should satisfy the condition:

$$\sigma_p \leqslant \frac{f_g}{\gamma_{R,g}} \tag{A.15.3-20}$$

with the representative interface transfer strength given by

$$f_p = \mu \left(\frac{P_{B,n}}{\pi/2\, D_p L_s} \right) \tag{A.15.3-21}$$

where μ is a generic coefficient of friction.

A value for μ between 0,1 and 0,2 may be used in the absence of specific data [A.15.3-2]; however, the use of elastomer lined clamps is not recommended, see 15.3.3 c) and 15.3.6.3.

A.15.3.5.6 Interface transfer strength of split sleeve clamps

No guidance is offered.

A.15.3.6 General requirements for bolted clamps

A.15.3.6.1 Mechanical clamps

The structural performance of mechanical clamps is sensitive to the fit-up achieved during installation, which in turn depends on the tolerances between the clamp and the structure. If a fit is too loose, the clamp can have substantially reduced strength. If a fit is too tight, the clamp can be installed incorrectly. Also, it is important that the contact between the clamp and the structure be as uniform as possible.

To achieve proper installation and effective performance of mechanical clamps, it is important to clean the surfaces to bare metal and to grind welds, seam or circumferential welds, flush with the substrate member outer diameter. Also, the clamp design should be based on an accurate dimensional survey of the actual structure.

Tests have shown [A.15.3-1] that corrosion products, formed immediately after blasting the substrate member(s) underwater, can affect performance. Therefore, mechanical clamps for underwater applications should be installed within 24 h of cleaning, ensuring that the surfaces are wiped clean of any corrosion products immediately before installation.

Generic flat-plate friction tests given in Reference [A.15.3-1] show that, for grit blasted steel surfaces exposed to salt air and water, the coefficient of friction varies between 0,4 and 0,57, with a mean value of 0,51.

A.15.3.6.2 Grouted clamps

No guidance is offered.

A.15.3.6.3 Lined clamps

Elastomeric materials, such as polychloroprene (commonly known as neoprene), have been used for lining the inside of friction repair clamps. The lining can be formed or notched to accommodate local irregularities (e.g. doubler plates) on existing members and can be bonded to the clamp saddle at room temperature.

Extreme care should be taken in the design and use of elastomer lined clamps. The design should consider the long-term effectiveness of the elastomer with regard to creep and sea water resistance. For example, soft and thick linings will have a higher initial deformation under force, and hence higher long-term creep, than hard and thin linings. Ribbed linings (linings with longitudinal grooves) should not be used for applications of primary force transfer, due to creep and their added flexibility.

A.15.3.6.4 Corrosion protection

No guidance is offered.

A.15.3.7 Bolting considerations

No guidance is offered.

A.16 Fatigue

A.16.1 General

A.16.1.1 Applicability

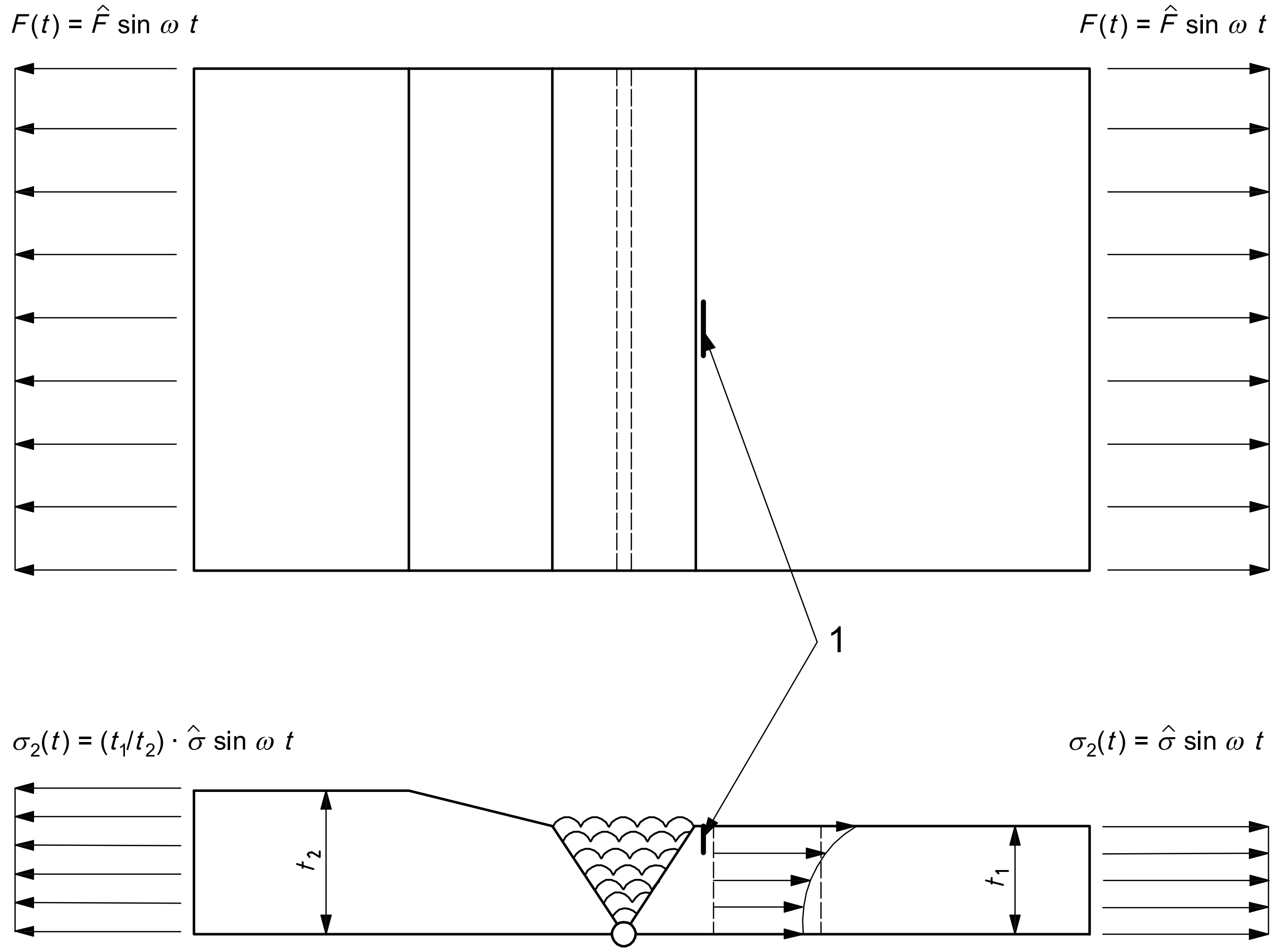
Fatigue has long been recognized as an important consideration for the design of offshore structures. Tubular joints in a fixed steel structure are especially subjected to large forces and are sensitive to fatigue damage;

intensive co-operative industry research on tubular joints occupied the full decades from the 1960s to the 1980s.

A.16.1.2 The fatigue process

Figure A.16.1-1 gives an idealized overview of the mechanics of the fatigue process. Its purpose is to assist basic understanding of the overall process and of the many separate factors involved. It further serves as a general reference for features and explanations of fatigue issues that are discussed in Clause 16. The basic description given here refers specifically to fatigue cracks emanating from pre-existing defects in welded connections, but the general concepts are equally applicable to fatigue in general.

Consider two plates of unequal thickness, connected by a full penetration butt weld perpendicular to the longitudinal direction of the plates; see Figure A.16.1-1. The plates are subjected to a variable action, $F(t) = \hat{F} \sin \omega t$, of constant amplitude and frequency. At sufficient distance away from the weld this action results in nominal stress distributions of constant amplitude, which are uniform over the cross-section of the plates. In the thinner plate the stress is $\sigma_1(t) = \hat{\sigma} \sin \omega t$, while in the thicker plate the stress is $\sigma_2(t) = (t_1 / t_2) \cdot \hat{\sigma} \sin \omega t$.



Key

1 small defect at weld toe

Figure A.16.1-1 — Fatigue illustration — Two plates of unequal thickness connected by a full penetration butt weld

At various locations in the body of the weld, or the heat affected zones in the plate material adjacent to the weld, small defects will inevitably exist. These defects will be very small, because large defects are avoided by proper design, workmanship and control of the weld process, or are captured by inspection during fabrication and repaired before the weld is accepted. The defects have in common that they are generally too small for detection, but otherwise they are due to different causes (e.g. lack of fusion, slag inclusions, shrinkage) and have different shapes, sizes and orientations. However, their most likely orientation will be more or less parallel to the weld. This orientation is in any case the most relevant and potentially the most onerous with a view to fatigue damage.

The defects represent crack-like features of microscopic size that are continuously subjected to variable stresses perpendicular to their most likely orientation. By way of example, a small defect at the toe of the weld is shown in Figure A.16.1-1. Under the influence of variable tensile stresses the defect will begin to grow in depth across the thickness of the plate, as well as in length across the width of the plate. The question is how many cycles of a particular magnitude the plate can withstand before it fails. To answer this question the following questions need first to be addressed.

a) **What is the magnitude and number of cycles of the nominal stress variations in the plate as a result of the applied action?**

The predominant source of time-varying actions on offshore structures is wave action. For flare towers and parts of topsides structures wind or mechanically induced vibrations can dominate. However, the present discussion will concentrate on wave-induced stress variations; hence wave action is the main driver of fatigue damage. The nominal stress in a component depends on the action effects (the internal forces) resulting from the applied actions (which vary with wave height, wave period and wave direction) and the structural response to the applied actions (which varies with type, location and orientation of the component, its boundary conditions, and other features). Wave conditions vary widely during the design service life of a structure; all these conditions should be taken into account. For the same wave condition the nominal stresses in different components will vary widely; all components should thus be checked for adequate resistance to fatigue. A fatigue assessment hence requires consideration of a large number of different load cases and a large number of different locations in the structure, and involves a very substantial computational effort.

b) **What is the local stress variation at the actual location of the initial defect?**

At the actual location of the initial defect the stress distribution will generally deviate from the uniform nominal stress distribution due to local variations of geometry; see Figure A.16.1-1. However, the stress raising effect caused by such changes of geometry at the location of the defect can be related to the nominal stress by the introduction of a so-called stress concentration factor (SCF). The SCF expresses the ratio of the local stress variation to the nominal stress variation and can both be larger and smaller than 1,0.

c) **Which stress parameter controls fatigue damage (e.g. the tensile stress amplitude, the ratio of minimum to maximum stress, the stress range between the maximum and the minimum stress in a cycle)?**

The actions on an offshore structure generally consist of a combination of permanent and time-varying actions. In welded connections the weld process and any constraints during the heat cycle additionally introduce residual stresses. The magnitude of these residual stresses is not known, but it is commonly accepted that tensile stresses of yield strength magnitude will exist around the weld. From this it logically follows that all, or at least the major part of, the full variable stress range will be in the tensile region. Therefore the stress range S is used as the controlling stress parameter.

d) **How can the resistance of the material (the number of stress cycles to failure) be determined?**

Crack growth due to time-varying (tensile) stresses can theoretically be described by the Paris-Erdogan law of fracture mechanics. However, the number of cycles that a material or a structural detail can withstand can generally only be empirically determined by testing the particular detail. The results of such tests are presented in terms of a pair of values of the applied stress range, S, and the number of cycles to failure, N. By repeating the tests for different values of S and counting the corresponding number of cycles N, a so-called S–N curve is determined, which defines the resistance of the material (or of the structural detail) to constant amplitude variable stresses of different magnitudes. The relationship between S and N has been found to be exponential and the S–N curve plots as a straight line on double logarithmic scales. Duplicating tests under the same circumstances (for so far as test conditions can be precisely controlled) demonstrates that the experimental outcome for a given value of S is not one unique value of N, but that significant scatter is observed. Therefore, the experimental results are presented as log S versus the mean of log N. To account for the uncertainty associated with this scatter, an S–N curve of log S versus the mean minus two standard deviations of log N is used for design purposes.

e) **What is the definition of failure?**

In view of the sensitivity of the test results it is of great importance to clearly define which definition of stress and which definition of failure is adopted. Definitions for both these parameters have been agreed upon internationally. The governing stress parameter is either the nominal stress range, $S_{nom} = \sigma_{max} - \sigma_{min}$ (which, in the present example, is equal to $S = 2\hat{\sigma}$), or a specially defined local stress range derived from $\sigma(t)$ as experienced at an identified location. The number of cycles to failure is defined as the number of cycles, N, for the crack to grow through the thickness of the test sample. In using and interpreting S–N curves for a fatigue assessment, these definitions should clearly be borne in mind. From the definition of N it further follows that S–N curves can be dependent on the thickness of the plate.

For geometries other than the simple example used here (for the purpose of illustration only) the definition of the stress parameter becomes more difficult. This is due to the more complex geometry of the structural components connected, as well as to the shape of the weld connecting them. In some cases the stress raising effect resulting from the geometry of the structural detail in combination with the type of local stresses can be rather well determined; in such cases preference is generally given to a local stress range. This is, for example, often the case for tubular joints, where the nominal stress range, $S_{nom} = \sigma_{max} - \sigma_{min}$, is multiplied by a geometrical SCF (C) so that $S = C \times S_{nom} = C(\sigma_{max} - \sigma_{min})$. The SCF differs with the type of forces to which the members that are connected at the joint are subjected (axial forces, in-plane bending or out-of-plane bending), with the structural details of the tubular joint, and with the location around the circumference of the connecting weld under consideration. However, in other cases the geometrical stress raising effect is not easily established, or it is much too cumbersome to determine for each and every different type of detail and load case; in such situations it is more convenient to use the nominal stress range $S_{nom} = 2\hat{\sigma}$. This latter case implies that a different S–N curve applies to each and every detail and load case.

It is, in any event, the stress-raising effect due to the geometry of the structural detail only that is included in the SCF. Additional stress raising effects at the location of the actual defect, which are due to the profile and internal structure of the weld (the so-called notch effect), are incorporated in the empirical S–N curve.

f) **How can the damage done be determined when the stresses are not variable stresses of constant amplitude?**

When the stresses in the structural component are not of the constant amplitude type, the number of cycles, n_i, of stress range class, S_i, that occur during a time period, T, are counted. The partial damage done by stress range class, S_i, is equal to the number of cycles, n_i, actually experienced divided by the corresponding number of cycles to failure, N_i. The total damage done is then the sum of the partial damages over all stress range classes; i.e. the total damage done is assumed to be equal to the non-dimensional number $D = \sum_i \frac{n_i}{N_i}$. Failure is assumed to occur when $D = 1{,}0$. This hypothesis of the linear accumulation of damage is known as the Palmgren-Miner rule. The calculated fatigue life, L, is the inverse of the total fatigue damage done multiplied by the time period T, i.e. $L = D^{-1} \times T = T/D$.

A.16.1.3 Fatigue assessment by analysis using S–N data

No guidance is offered.

A.16.1.4 Fatigue assessment by analysis using fracture mechanics methods

No guidance is offered.

A.16.1.5 Fatigue assessment by other methods

No guidance is offered.

A.16.1.6 Fatigue assessment of existing components

No guidance is offered.

A.16.2 General requirements

No guidance is offered.

A.16.3 Description of the long-term wave environment

A.16.3.1 General

Ocean waves are irregular in shape, vary in height, length and speed of propagation, and can approach a structure from one or more directions simultaneously. During a sea state these features of a real sea can be described by means of a random wave model. The linear random wave model views the sea as the superimposition of many small individual frequency components, each of which is a periodic wave with its own amplitude, frequency and direction of propagation, and which have random phase relationships with respect to each other. A unidirectional random sea is a special case of this in which all frequency components propagate in the same direction. The surface of a unidirectional sea is hence long-crested, whereas the surface of a real or directional sea is short-crested. In the linear random wave model, a unidirectional sea is completely described by the two-parameter frequency spectrum, $S(\omega)$; see ISO 19901-1.

A.16.3.2 Wave scatter diagram

For the representative period, T_{R}, of a sea state several choices are in common use. T_{R} may be chosen to be the peak spectral period, T_{p}, or the average zero-crossing period, T_{z}, or an alternative mean period, T_1; see ISO 19901-1. These periods are distinctly different and it is important to ensure that the period by which the data are given and the period that is used in the analysis are consistent.

Wave scatter diagrams usually refer to a (typical) year and are normally given in a normalized form of number of occurrences per thousand observations. Generally available data usually refer to two-dimensional wave scatter diagrams supplemented with a distribution of mean wave directions, which is often assumed to be equal to that of the mean wind directions. An example of a two-dimensional wave scatter diagram is shown in Figure A.16.3-1.

See ISO 19901-1 for guidance on the determination and adequacy of the metocean database.

Significant wave height (m) \ Mean zero-crossing period (seconds)	2,0–3,0	3,0–4,0	4,0–5,0	5,0–6,0	6,0–7,0	7,0–8,0	8,0–9,0	9,0–10,0	10,0–11,0	Totals
Totals	631	6 855	14 500	12 802	6 227	1 235	151	11	1	42 413
7,5–8,0									1	1
7,0–7,5								1		1
6,5–7,0							4	2		6
6,0–6,5						5	18	1		24
5,5–6,0						31	69	2		102
5,0–5,5					2	159	41			202
4,5–5,0					65	437	3			505
4,0–4,5				3	679	391				1 073
3,5–4,0				53	2 250	54	1			2 358
3,0–3,5				1 352	2 171	24				3 547
2,5–3,0			27	4 810	386	58		1		5 282
2,0–2,5			2 204	4 283	374	44	8	2		6 915
1,5–2,0		20	6 418	1 594	216	19	4	1		8 272
1,0–1,5		3 042	4 950	622	72	7	1			8 694
0,5–1,0	324	3 492	870	77	10	3	1	1		4 778
0–0,5	307	301	31	8	2	3	1			653

Figure A.16.3-1 — Example of two-dimensional wave scatter diagram

A.16.3.3 Mean wave directions

See ISO 19901-1 for a general discussion of wave conditions.

A.16.3.4 Wave frequency spectra

See ISO 19901-1 for detailed information on waves and wave spectra.

A.16.3.5 Wave directional spreading function

Experience has shown that in most cases the influence of wave directional spreading around the mean is of secondary importance and can be neglected. However, if it is considered that the influence of wave directional spreading on the stress variations is of some significance and it is desired that this effect is included, this can be done by choosing an appropriate wave spreading function in accordance with ISO 19901-1.

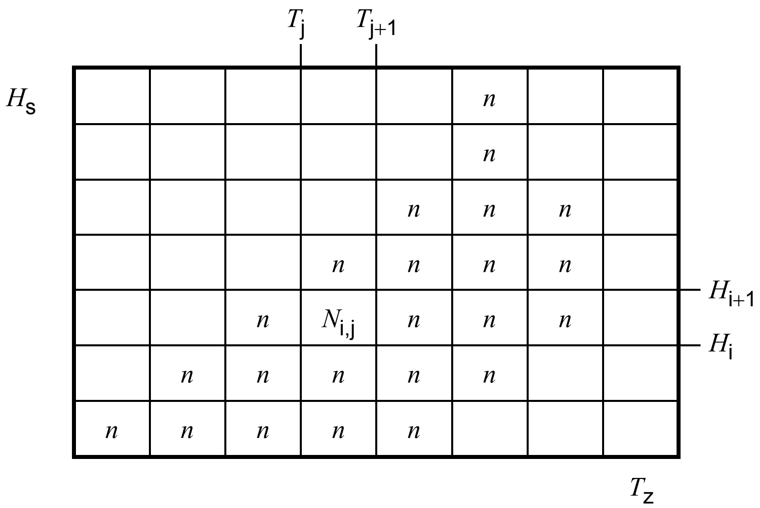
A.16.3.6 Periodic waves

The deterministic waves should be defined in accordance with ISO 19901-1.

A.16.3.7 Long-term distribution of individual wave heights

Figure A.16.3-2 is a schematic of a wave scatter diagram in which the sea states are defined by the significant wave height, H_s, and the average zero-crossing period, T_z. If the representative period, T_R, of the data is not the average zero-crossing period, T_R should first be converted into T_z. The selected sea state indicated in the Figure occurs $N_{i,j}$ times out of a total of N_T observations, where i is the subscript for significant wave height and j the subscript for representative period and

$$N_T = \sum_j \left(\sum_i N_{i,j} \right) = \sum_i \left(\sum_j N_{i,j} \right)$$ is the total number of observations in the diagram.



Key

H_s significant wave height

T_z mean zero-crossing period

$N_{i,j}$ number of occurrences of sea states with $H_i < H_s < H_{i+1}$ and $T_j < T_z < T_{j+1}$

n number of occurrences of sea states falling within indicated range of H_s and T_z

Figure A.16.3-2 — Example of wave scatter diagram

The probability that the individual wave height, H, in a sea state with significant wave height, H_s, has a particular value, H^*, is given by $p(H^*)\mathrm{d}H$, where $p(H^*)$ is the Rayleigh probability density function:

$$P(H^*) = \frac{4H^*}{H_s^2} \cdot \exp\left[\frac{-2(H^*)^2}{H_s^2} \right] \tag{A.16.3-1}$$

The cumulative probability, $p(H > H^*)$, that the individual wave height, H, exceeds value H^* in the sea state with significant wave height H_s is thus

$$P(H > H^*) = \int_{H^*}^{\infty} p(H)\mathrm{d}H = \int_{H^*}^{\infty} \frac{4H}{H_s^2} \cdot \exp\left(\frac{-2H^2}{H_s^2} \right) \cdot \mathrm{d}H = \exp\left[\frac{-2(H^*)^2}{H_s^2} \right] \tag{A.16.3-2}$$

The number of times in any one year that individual wave height H^* is exceeded in the selected sea state, i, j, is indicated by the symbol $(N_{H^*})_{i,j}$ and given by

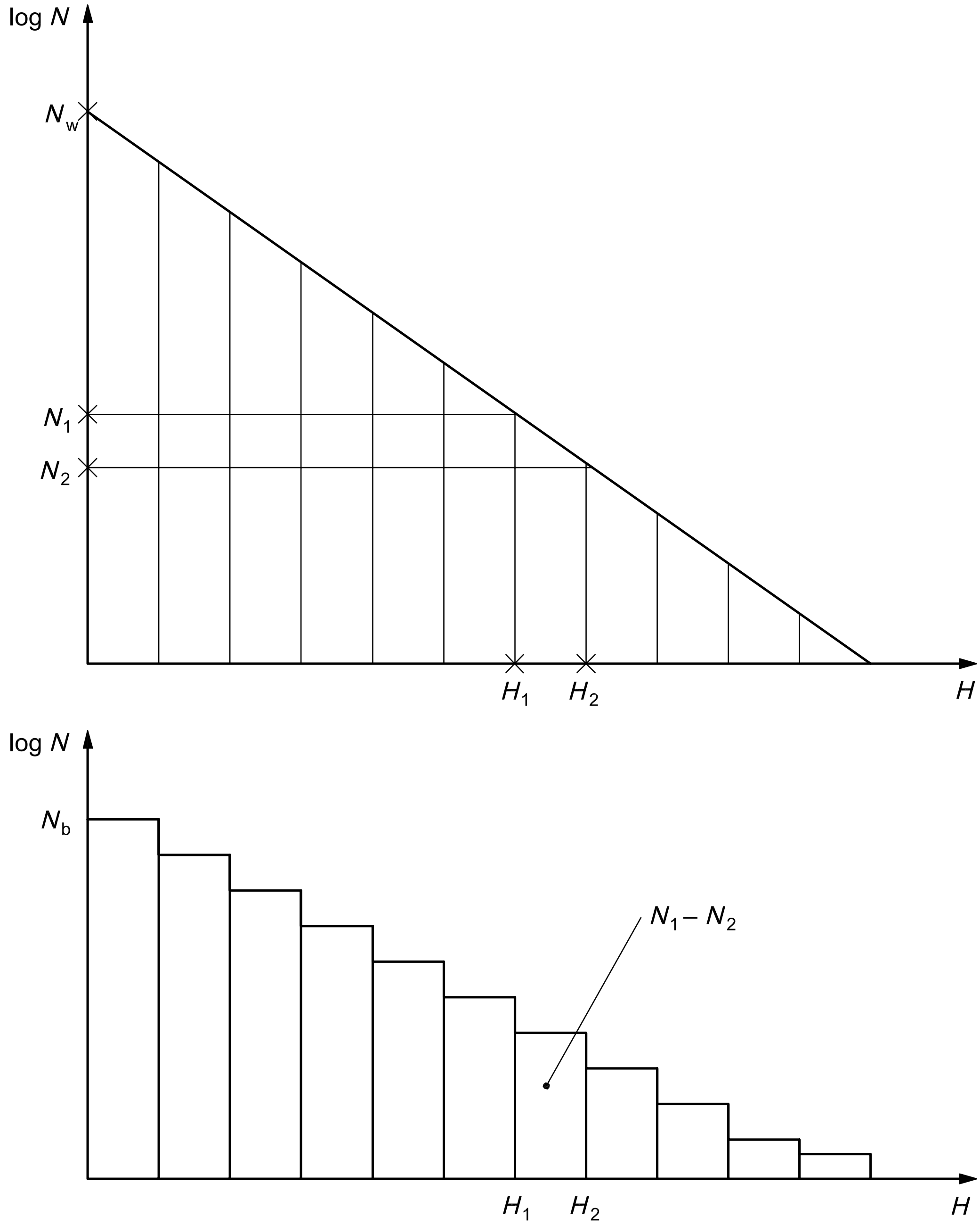
$$\left(N_{H^*}\right)_{i,j} = \left(\frac{N_{i,j}}{N_T}\right) \cdot \left(\frac{(365)(24)(3\,600)}{(0{,}5)(T_{z,j} + T_{z,j+1})}\right) \cdot P(H > H^*) \qquad \text{(A.16.3-3)}$$

The total number of exceedances per year, N_{H^*}, of the individual wave height, H^*, is found by repeating this calculation for all sea states in the scatter diagram and summing the results:

$$N_{H^*} = \frac{(365)(24)(3\,600)}{N_T} \cdot \sum_i \left\langle \exp\left\{ \frac{-2\left(H^*\right)^2}{\left[(0{,}5)(H_{s,i} + H_{s,i+1})\right]^2} \right\} \sum_j \frac{N_{i,j}}{(0{,}5)(T_{z,j} + T_{z,j+1})} \right\rangle \qquad \text{(A.16.3-4)}$$

The long-term cumulative distribution of individual wave heights H is obtained by repeating the calculation of Equation (A.16.3-4) for all values of H^*. The total number of waves per year N_W is the sum of N_{H^*} for all values of H^* for which the calculations are made.

Figure A.16.3-3 shows a typical example of the long-term cumulative distribution of individual wave heights and its discretization into wave height blocks. It is usually close to an exponential distribution that plots approximately as a straight line when plotted on log-linear scales.



Key

H	individual wave height (on linear scale)
N_w	total number of individual waves (on logarithmic scale)
N_b	number of waves in a block
H_1	wave height 1
H_2	wave height 2
N_1	number of waves exceeding H_1
N_2	number of waves exceeding H_2

Figure A.16.3-3 — Typical long-term cumulative distribution of individual wave heights

A.16.3.8 Current

In design practice for strength design it is common to use an in-line design current in combination with a design wave. This is not appropriate for a fatigue analysis. Each combination of H_S and T_R in the wave scatter diagram represents a large number of occurrences of different sea states. All these occurrences have the same statistical parameters of H_S and T_R, but are in other respects likely to be (very) different. Currents usually vary widely over time in both speed and direction, and no specific current information can normally be associated with each individual sea state.

Furthermore, for a fatigue assessment it is not the absolute value of the stress that matters but the stress range. Therefore it is also the range rather than the absolute magnitude of the applied actions that should be considered.

The presence of a current in conjunction with waves affects the drag component of the hydrodynamic actions on the structure from the Morison equation in two ways, i.e. it will produce a mean value of the action and it will modify the variable part of the wave and current action, compared to the variable part of the action in waves alone. The mean value is not relevant for fatigue. For an in-line current the maximum action under the wave crest and the minimum action under the wave trough are both increased. Therefore, the range of the applied action (the maximum minus the minimum action) is much less affected than the peak action in an absolute sense. For current directions that are not in-line with the wave, the influence is even smaller.

Sensitivity analyses have also shown that the influence of currents on the range of the calculated stress variations and hence on the fatigue damage accumulation in the structure is normally rather small.

In view of the above it is not only impractical, but normally fully justified, to neglect current in calculating the applied wave actions for fatigue analyses. The remaining uncertainty is considered to be included in the additional fatigue damage design factor and/or the field experience factor; see 16.2.8 and 16.12.

A.16.3.9 Wind

No guidance is offered.

A.16.3.10 Water depth

In contrast with analyses for strength design in extreme environmental conditions, mean sea level should be used for a fatigue analysis, accounting for any settlement of the structure where appropriate. If, additionally, sea floor subsidence is expected to occur, the water level used in the analysis should reflect the average subsidence during the design service life; if the expected subsidence is substantial, several mean sea levels should be considered.

A.16.3.11 Marine growth

Marine growth development is usually most pronounced during the first few years after installation, after which a more or less stable situation arises. The design marine growth thickness is the marine growth that is expected to accumulate on the structure without being removed. It therefore represents a situation that can exist during the entire design service life and which should be taken into account in a fatigue assessment.

For the fatigue assessment of an existing structure, the actual marine growth distribution on the structure can be used, provided this can reliably be established and shown to be stable.

A.16.4 Performing the global stress analyses

A.16.4.1 General

No guidance is offered.

A.16.4.2 Actions caused by waves

To determine the stress range during the passage of a wave, a full periodic wave cycle should be stepped past the structure. The number of wave positions should be sufficient to accurately determine the maximum and the minimum stress during the passage of the wave. The definition of a full stresses cycle requires, in general, a substantial number of points. For 12 equally spaced points in a perfectly sinusoidal variation, the peak can still be missed by 15°; as cos 15° = 0,966, the error in the true stress range can amount to 3,4 %. As fatigue damage is governed by the stress range to the power of the slope of the S–N curve, this can easily introduce an error of some 10 %, i.e. $(1{,}034)^3 \approx 1{,}10$.

Each calculated point in the stress cycle corresponds with a wave position, thus requiring the calculation of distributed wave actions over the full structure and a subsequent structural analysis for each wave position. Some computer programs therefore perform interpolation of the stress cycle between calculated points in a post-processing operation, by which the number of individual wave positions and associated structural analyses can be reduced. The interpolation procedure used for this purpose should at least be of second order.

If the marine growth thickness is such that certain assemblies of components (e.g. conductor guide frames, fenders or boat landings) are completely blocked, then the associated wave action will be substantially increased. If this is likely to occur, its effect should be properly incorporated in modelling the hydrodynamic actions on these components.

As the periodic waves used for fatigue analyses are low and short compared to design waves for a strength analysis, the associated Keulegan-Carpenter numbers, $K = U_\mathrm{m}\, T/D$ (see A.9.5.2.3), are also much lower. They are largest at the free water surface. Assuming deep water waves, the maximum attainable value is $K = \pi H/D$, while the value decreases exponentially through the water column with depth below the surface. The corresponding value of the inertia coefficient is therefore larger than that recommended for the design wave analysis. For many combinations of waves and typical member sizes, the inertia coefficient will be approximately equal to 2,0. The theoretical value of the inertia coefficient for negligible wake effects around members is also 2,0. For these reasons, $C_\mathrm{m} = 2{,}0$ is adopted for fatigue analyses.

A.16.4.3 Quasi-static analyses

The dynamic amplification of quasi-static stress responses is governed by the magnitude and the frequency range of the excitation of the structure in relation to the platform natural frequencies. Wave periods that coincide with structural natural periods of 2,5 s and below correspond with wave lengths of approximately 10 m and less. While these can still be effective in producing local wave actions on typical member sizes of space frame structures in accordance with Morison's equation, global action on the whole structure rapidly diminishes with increasing frequency due to partial cancellation resulting from phase differences between the local actions on spatially distributed members.

For monotowers, or configurations with highly correlated members, such partial cancellations do not occur. Therefore, the decrease in global wave action on such structures can only be due to wave diffraction effects for wave length to diameter ratios of less than approximately 5. For such configurations, the neglect of dynamic effects should be judged on the basis of the magnitude of the excitation at the platform natural frequencies.

A.16.4.4 Dynamic analyses

A.16.4.4.1 General

For calculating global structural responses, modal analysis techniques can offer an efficient and reliable method in some circumstances. However, for local responses, pure modal techniques are usually unreliable.

Unless a very large number of modes are included, a modal solution will not accurately represent the local (quasi-static) member deformations, which are important for a fatigue analysis. Choosing a very large number of modes is unlikely to improve the situation with confidence due to numerical inaccuracies associated with the high frequency modes. To overcome both of these aspects, the so-called *method of modal analysis plus static back substitution* has been developed, which superimposes the dynamic responses due to a limited number of modes onto a full static solution. The method is also known by the name *mode acceleration method*. This method offers the benefits of a combination of accuracy and computational efficiency. When using this procedure, it is still necessary to include a sufficient number of modes to accurately represent all significant dynamic responses.

However, a direct solution technique is generally preferred over a modal type of solution, provided this solution method is available in the structural analysis software used. When using a direct solution technique, a modal analysis should also be performed to obtain natural frequencies and mode shapes for the principal modes of the structure. This serves generally to verify that the dynamic behaviour of the model is realistic, and provides the values of the natural frequencies that are used in the selection of the frequency grid as described in 16.7.2.2. Further particulars of the above alternative solution techniques are described in Reference [A.16.4-1].

If a natural frequency falls in a valley in the transfer function of the applied wave action it should be shifted to a more conservative location. A realistic range of natural frequency values should be considered, based on lower and upper bound values for the platform mass and stiffness as discussed in A.16.4.4.2 and A.16.4.4.3. The choice of which parameter to adjust for achieving such a shift is structure-specific and depends upon deck mass, soil conditions and structural configuration. It should be recognized that adjusting the foundation stiffness can unintentionally alter the member forces in the base of the structure, which can be fatigue sensitive.

A.16.4.4.2 Mass

For slender members, the added mass should be based on the displaced volume for motion transverse to the longitudinal axis, with an appropriate added mass coefficient to account for the member shape. Guidance on added mass coefficients can be found in the literature. For complex assemblies the added mass should be determined for each motion direction and should account for the actual geometry. Account should further be taken of the increase in volume due to marine growth as well as possible blocking effects associated with marine growth.

The equipment and consumable masses in the model should include all items supported by the structure during any given operation on the platform. If the mass of such items is predicted to vary significantly for different operational phases during the structure life, it is appropriate to perform separate analyses for each phase and combine the associated fatigue damage.

The masses that are used in the model should be reconciled with the topsides and structure weights; centres of gravity should be established with the weight-estimating procedure used. When assessing the range of natural period values that can occur, upper and lower bound estimates of the masses should be considered. The lower bound mass should be determined with marine growth omitted.

A.16.4.4.3 Stiffness

The structural model of a space frame structure consists primarily of beam elements and the modelling detail should be adequate to provide accurate stresses at all relevant connections. The member intersections should be modelled such that the resulting nominal member end stresses are consistent with their subsequent use in the fatigue analysis. For typical space frame members, nominal brace stresses are normally computed at the intersections of the brace and chord centre lines. For large diameter chords or short braces, local joint geometry can require more refined modelling.

For stiffness modelling, when determining the wall thickness of members that include a corrosion allowance, it should normally be assumed that half of the allowance has been consumed, to represent the average condition during the life of the member.

The stiffness of appurtenances (conductors, caissons, risers, J-tubes, boat landings, fenders, launch rails, skirt piles, pile-to-sleeve connections, mudmat framing, etc.) should be included if their connection details contribute to the overall stiffness and transfer of forces in the structure. The transfer of forces at support points should be realistically modelled. Detailed stiffness modelling of appurtenances is also required if a fatigue analysis of the appurtenances themselves is to be performed.

Complex assemblies such as pile-to-sleeve connections, mudmats, conductor guides, etc., can require finite element types other than beam elements (e.g. shell, plate or solid elements). Detailed finite element analyses of such assemblies may be performed separately from the global analysis. These may be used to determine equivalent stiffness properties for simplified beam element representations for inclusion in the global stiffness model and can also assist in fatigue checking of the assembly details. Alternatively, some computer programs allow the use of super elements within the global stiffness model for representing such complex assemblies.

A.16.4.4.4 Damping

A relative velocity formulation of the Morison equation inseparably links wave excitation and hydrodynamic drag damping. It reduces applied wave actions and causes implicit hydrodynamic damping associated with drag actions, which is additional to any damping from a viscous damping coefficient introduced in accordance with 16.4.4.4. For small structural displacements, as are typical for fatigue excitation, this additional damping is not observed in measurements and is inappropriate. Consequently, the relative velocity formulation should not be used for a fatigue analysis.

A.16.5 Characterization of the stress range data governing fatigue

In this International Standard, the terms *geometric stress* (GS) and *geometric stress range* (GSR) are used, replacing the terms hot spot stress and hot spot stress range, which were previously commonly used. These new terms have been introduced to emphasize that stress concentrations occur to a varying degree all along the welds and not only at specific locations.

The nominal axial stress at the member end is denoted by $\sigma_{\mathrm{ax}}(t)$; the nominal in-plane bending stress is denoted by $\sigma_{\mathrm{ipb}}(t)$ and is assumed to be due to the moment, M_{y}, about the local member y-axis; the nominal out-of-plane bending stress is denoted by $\sigma_{\mathrm{opb}}(t)$ and is assumed to be due to the moment about the local member z-axis. The geometric stress, $\sigma_{\mathrm{GS,A}}(t)$, at a specific location, A, around the tubular connection is then determined using Equation (A.16.5-1), see Figure A.16.5-1:

$$\sigma_{\mathrm{GS,A}}(t) = C_{\mathrm{ax}}\ \sigma_{\mathrm{ax}}(t) \pm C_{\mathrm{ipb}}\ \sigma_{\mathrm{ipb}}(t)\sin\theta \pm C_{\mathrm{opb}}\ \sigma_{\mathrm{opb}}(t)\cos\theta \qquad \text{(A.16.5-1)}$$

where the signs in the formula are determined by the sign conventions employed in the analysis. The range of $\sigma_{\mathrm{GS,A}}(t)$ is the GSR (S). Due to phase differences between the various stress components, the GSR is normally not equal to the algebraic sum of the stress components.

For location A, at least the four main points around the circumference for $\theta = 0°$, 90°, 180° and 270° (the four main clock positions) should be considered. Further details of GSRs for tubular and other joints are discussed in A.16.10.

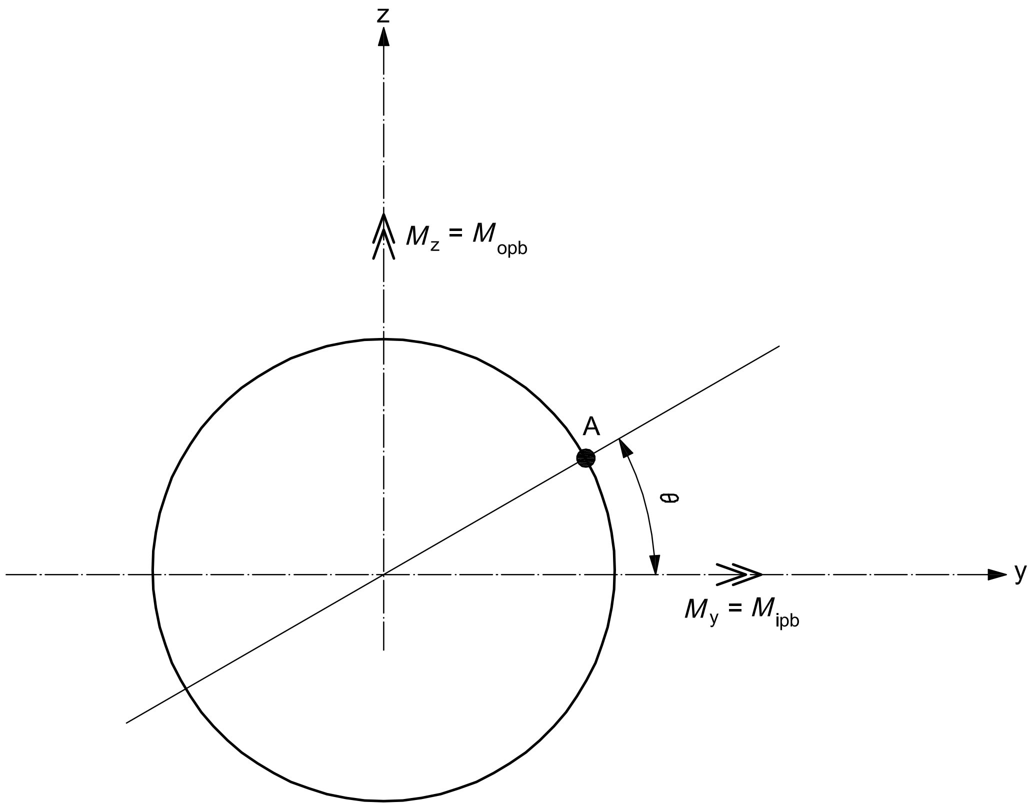


Figure A.16.5-1 — Determination of geometric stress at point A around tubular connection

A.16.6 The long-term local stress range history

No guidance is offered.

A.16.7 Determining the long-term stress range distribution by spectral analysis

A.16.7.1 General

Spectral analysis is an established technique for the prediction of the responses of a linear system to random excitation, see, for example, References [A.16.7-1] and [A.16.7-2]. Its application to fatigue assessments in the offshore environment is described in, e.g. References [A.16.7-3] and [A.16.7-4]. The method has been applied to a wide range of offshore structures of various types in various environments.

A.16.7.2 Stress transfer functions

A.16.7.2.1 General

A flow chart for the spectral analysis procedure including the calculation of the stress transfer function is shown in Figure A.16.7-1.

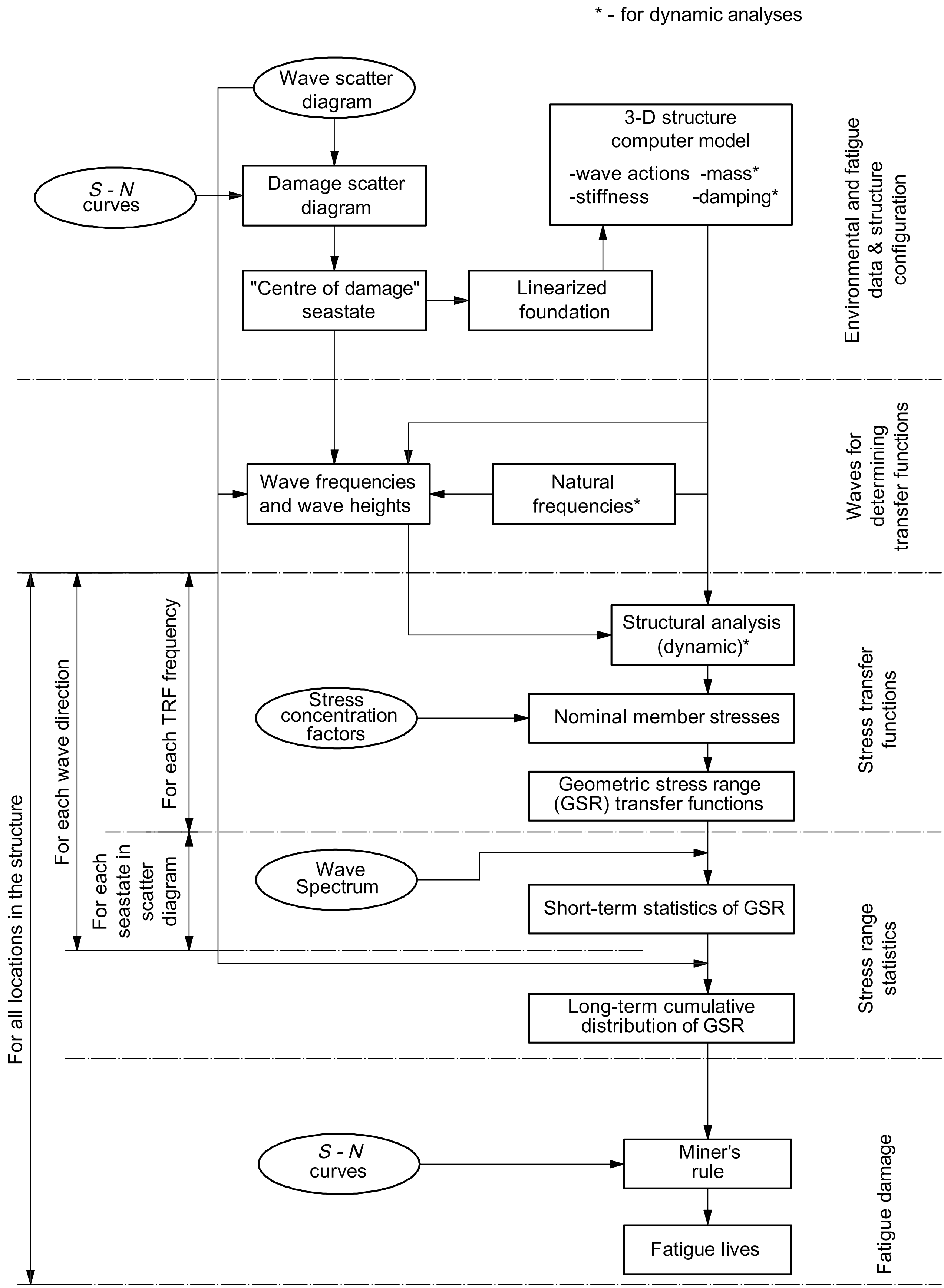


Figure A.16.7-1 — Flow chart of spectral analysis procedure

For a spectral analysis, the GSR transfer functions are required for all locations in the structure that should be checked for fatigue. These functions define the GSR per unit wave height over a range of wave frequencies for each wave approach direction. A GSR transfer function is determined as follows.

For a given wave approach direction and a given wave frequency a global structural analysis is performed using a periodic (regular) wave. If the analysis is performed in the frequency domain, a linear wave theory should be selected. The wave height is not relevant and usually a unit wave amplitude (or wave height) is chosen. If the analysis is performed in the time domain, a non-linear wave theory may be chosen; in this case the wave height is selected in accordance with 16.7.2.3. Further guidance on the global structural analysis is given in 16.4.

At each location in the structure and for each relevant stress component the nominal stress and the associated phase with respect to the wave is determined. Using these stress components, the geometric stress $\sigma_{GS,A}(t)$ at point A is determined as described in 16.5. The range of $\sigma_{GS,A}(t)$ is the GSR at point A. Dividing the GSR by the wave height, one point of the GSR transfer function for point A is defined; from this moment onwards, the phase angle information can be dropped in the further processing. This procedure is repeated for all points A around the connection and all locations in the structure.

The above calculations are next repeated for all wave frequencies in order to fully define the GSR transfer functions for one particular wave approach direction; subsequent to that, they are repeated for all wave approach directions.

A.16.7.2.2 Selection of wave frequencies

Figure A.16.7-2 shows an example of the shapes of a transfer function (TRF) of static and dynamic actions on the structure due to waves and depicts several typical features as a function of frequency.

The following guidance is offered for the selection of wave frequencies in order to take the characteristics of the environment as well as the characteristics of the structure duly into account.

a) Lowest and highest frequency

The lowest and highest frequency used should be based on the range of frequencies over which there is significant energy in the sea states in the wave scatter diagram. For dynamic analyses, special attention should be paid to the high frequency cut-off.

b) Cancellation and addition frequencies

Wave frequencies in the vicinity of peaks and troughs in the transfer functions of applied wave action (curve A in Figure A.16.7-2) should be included to ensure that the structural response at these frequencies is properly defined. For simple structures, the frequencies at which addition and cancellation of wave actions on structural components occurs can be reasonably predicted, as they correspond with wave lengths that are specific multiples or fractions of key dimensions of the structure, such as the main leg spacing. For complicated structures these features are often not so obvious from the structural drawings and it can be necessary to determine the corresponding frequencies by inspection of the calculated wave actions and/or the wave induced responses.

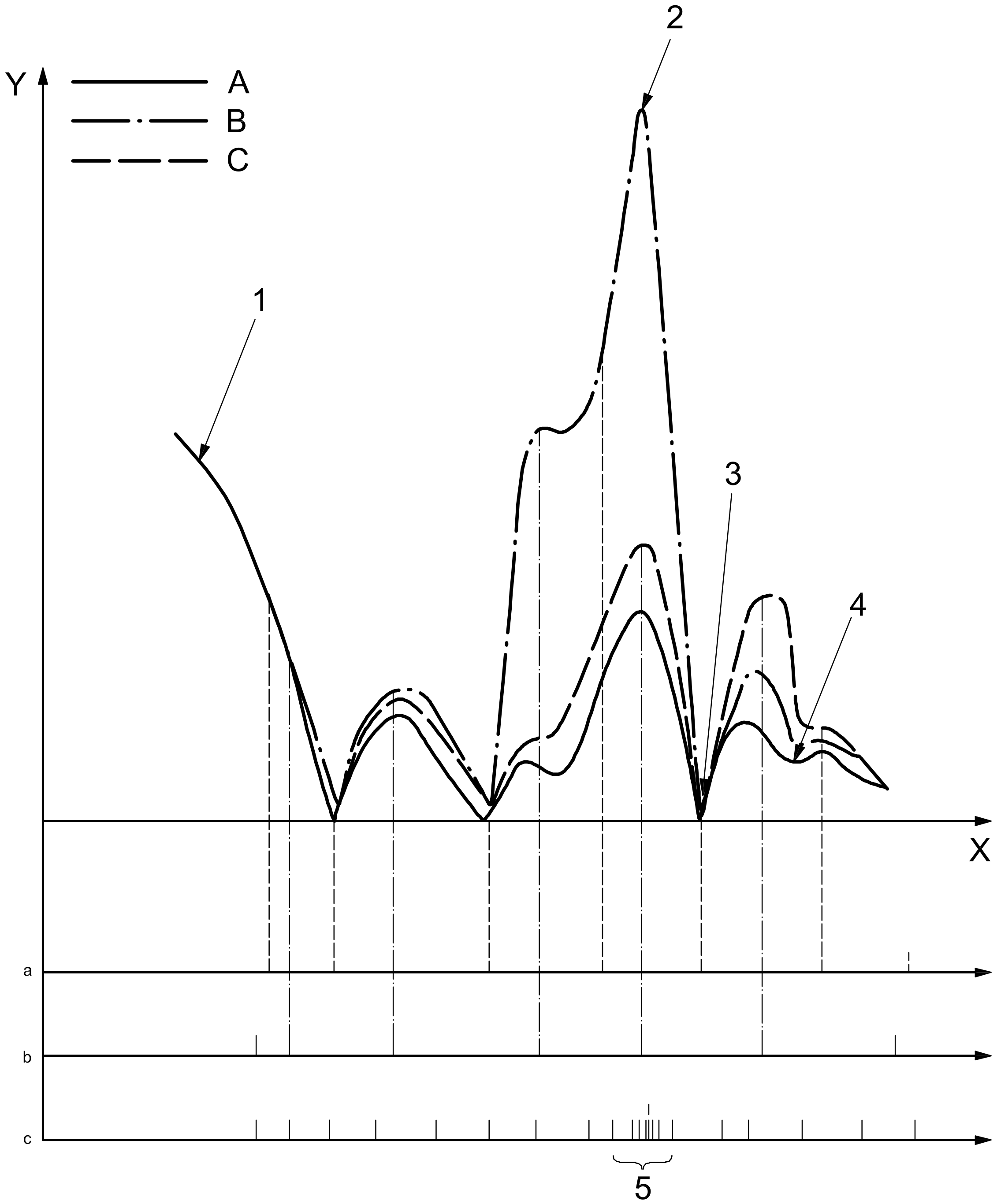
c) Intermediate frequencies

A sufficient number of intermediate frequencies should be included, in order to ensure a complete definition of the shape of the transfer function. To reduce the number of stress analyses required, some computer programs perform interpolation to derive transfer function ordinates at intermediate frequencies. However, care should be exercised with such interpolation routines due to the often complex shape of the transfer functions. Interpolation is more reliable when this is performed on the real and imaginary parts (the in-phase and out-of-phase components) of the transfer function separately, because these are normally smooth functions.

Selection of intermediate frequencies corresponding with peaks of the wave spectra in the scatter diagram is further necessary in order to provide accurate definition of wave input energy.

d) Natural frequencies

For dynamic analyses, the natural frequencies of the structure should also be included (curves B and C in Figure A.16.7-2). In addition, a number of closely spaced frequencies on both sides of a natural frequency should be included so as to accurately define the sharp dynamic response peak of a lightly damped system over a frequency range of $(1 \pm 4\beta)\ \omega_n$ around resonance, where ω_n is the natural frequency in radians per second and β is the fraction of critical damping. The spacing of these frequencies should preferably be 0,5 $\beta\omega_n$, and definitely not greater than $\beta\omega_n$.



Key

A quasi-static transfer function

B dynamic transfer function — resonance at "peak"

C dynamic transfer function — resonance at "valley"

X wave frequency, ω

Y normalized applied wave action at wave frequency, E_w

1 low frequency, long waves, no cancellation effects

2 structure natural frequency at peak in static TRF

3 structure natural frequency at valley in static TRF

4 peaks and valleys due to interaction between wave length and structure geometry

5 fine grid around natural frequency

Frequency selection

[a] Poor: underpredicts response.

[b] Poor: overpredicts response.

[c] Good: adequately represents TRF.

Figure A.16.7-2 — Typical transfer functions of total applied actions due to waves

A.16.7.2.3 Selection of wave heights

A.16.7.2.3.1 Determination of wave height

The linearization for an analysis in the time domain is performed by the rational selection of wave heights used in the regular wave analyses for the determination of the stress transfer functions. Alternative approaches can be used for the selection; one well tested method is described below.

Calibration of the wave steepness should be based on the matching of global response parameters, which are representative of the predominant wave action to which the structure is subjected under fatigue conditions. For typical space frame structures, the total applied wave action (quasi-static base shear) is normally used for this purpose. The matching should be performed for the sea state at the centre of the fatigue damage scatter diagram; the derivation of this sea state is described in A.16.7.2.3.2.

The calibration determines a wave steepness value, which matches the spectrally calculated range of applied wave action to the deterministically calculated range of applied wave action for the structure under consideration and the sea state at the centre of the fatigue damage scatter diagram. The calibration should be performed for various wave directions. Typically, a broadside, an end-on and a diagonal wave direction are considered as a minimum. The calibration process consists of the following five steps.

a) Determine the transfer function of the total applied wave action (quasi-static base shear) for each wave direction and for a range of wave steepness values. Note, however, that use of a constant wave steepness gives unrealistically large wave heights at low wave frequencies (large wave lengths). Therefore, the wave heights at the low frequency side should be capped. A wave height equal to the wave height with a one year return period should normally be used as a maximum.

b) Calculate the root-mean-square (rms) value of the applied wave action for each wave direction and the sea state at the centre of the fatigue damage scatter diagram (see A.16.7.2.3.2), using standard spectral analysis techniques. This provides rms values of the applied wave action for each wave direction and the initially chosen wave steepness values.

   The linearized spectral response is a Gaussian process and, assuming a standard duration of some three hours with approximately 1 000 response peaks, the most probable maximum (MPM) values of the applied actions due to waves are 3,7 times the rms values. In this manner, the MPM values of the applied wave action from the spectral calculations are determined for each wave direction and wave steepness. The applied ranges of wave action that should be used in the calibration are twice the MPM values of the applied wave action.

c) Next, a deterministically calculated range of applied wave action is also determined by stepping a deterministic wave through the structure for each wave direction. This deterministic wave should have a height that is equal to the MPM wave height in the sea state at the centre of the fatigue damage scatter diagram, and a period that corresponds with the peak period of the wave spectrum for this sea state. The deterministic range of applied wave action is the maximum minus the minimum total applied wave action during a full wave cycle.

d) For each wave direction, the calibrated value of the wave steepness is the steepness that matches the spectrally calculated MPM value of the range of applied wave action from b) to the deterministically calculated range of applied wave action from c).

e) From the set of calibrated wave steepness values for all wave approach directions obtained from d) a single representative wave steepness value should next be chosen.

The wave height for each wave frequency is now determined using the calibrated wave steepness value. The wave heights at the low frequency side should again be limited to the maximum wave height used in the calibration process. The resulting wave heights should further be compared to the elevations of plan framing levels near the water surface, to assess if any unreasonable discontinuities in the applied wave action are likely to occur. Where appropriate, some final adjustment should be made.

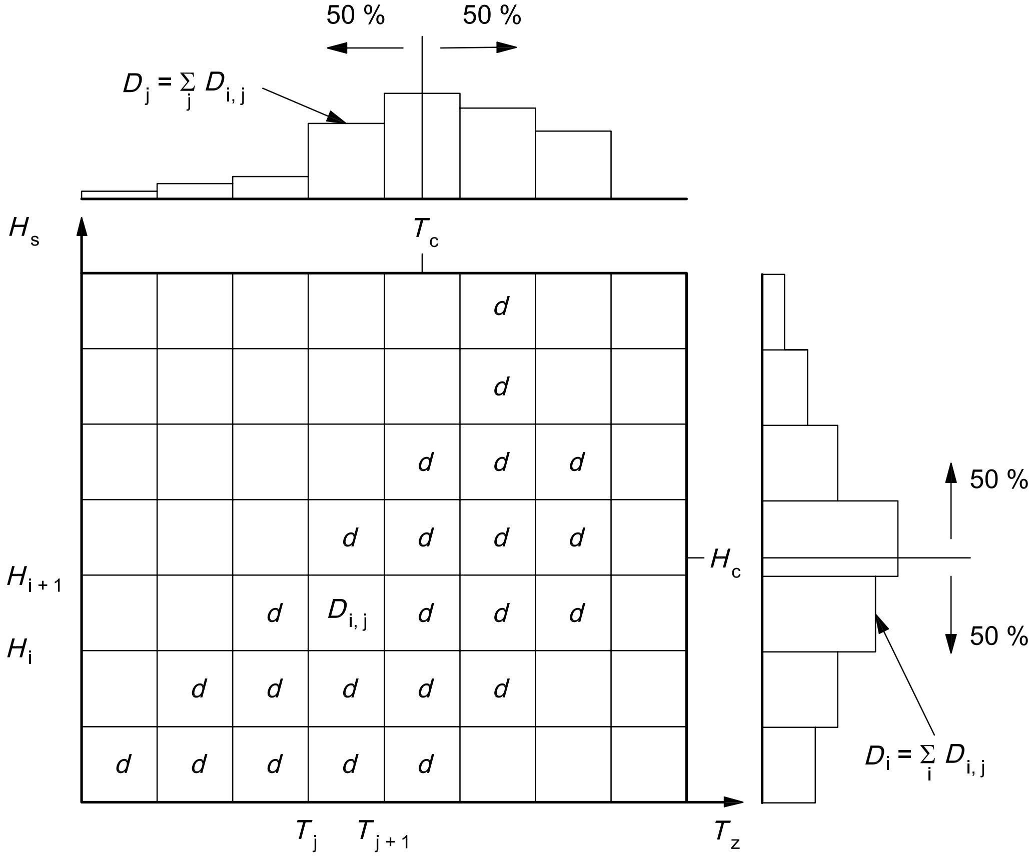
A.16.7.2.3.2 Derivation of the sea state at the centre of the fatigue damage scatter diagram

Stress is assumed to be a linear function of wave height, while the number of stress cycles can initially be assumed to be equal to the number of wave cycles in a sea state. Fatigue damage can further be assumed to be proportional to the stress range raised to the power of the S–N curve slope.

The partial damage, $D_{i,j}$, caused by a particular sea state is hence proportional to the number of occurrences of the sea state, $N_{i,j}$, and the significant wave height, H_s, raised to the power (m) of the slope of the S–N curve. Proportionality to the number of stress cycles in the sea state translates into an inversely proportional relationship to the mean zero crossing period, T_z. Consequently:

$$D_{i,j} \propto \frac{N_{i,j}\left[0,5\left(H_i + H_{i+1}\right)\right]^m}{0,5\left(T_j + T_{j+1}\right)} \tag{A.16.7-1}$$

The above calculation is repeated for each sea state in the wave scatter diagram to produce a damage scatter diagram with relative damages in the sea state bins as shown in Figure A.16.7-3.



Key

T_z mean zero-crossing period
H_s significant wave height
T_c central value of the mean zero crossing period
H_c central value of the significant wave height
$D_{i,j}$ fatigue damage from sea states with $H_i < H_s < H_{i+1}$ and $T_j < T_z < T_{j+1}$
D_i fatigue damage from sea states with $H_i < H_s < H_{i+1}$
D_j fatigue damage from sea states with $T_j < T_z < T_{j+1}$
d fatigue damage from sea states falling within indicated range of H_s and T_z

Figure A.16.7-3 — Typical fatigue damage scatter diagram

The sea state at the centre of the damage scatter diagram is defined as having

— a significant wave height, H_c, such that 50 % of the damage occurs in sea states with a lower H_s and 50 % with a higher H_s, and

— a mean zero crossing period, T_c, such that 50 % of the damage occurs in sea states having a lower T_z and 50 % with a higher T_z.

The most probable maximum wave height, H_{ref}, and the associated wave period, T_{ref}, for this centre of damage sea state are calculated as

$$\begin{aligned} H_{ref} &= (1{,}86) H_c \\ T_{ref} &= T_c / (0{,}81) \end{aligned} \qquad \text{(A.16.7-2)}$$

For linear $S–N$ curves on a log-log plot, these calculations can be performed analytically and independent of the magnitude of the stress response. However, for bilinear $S–N$ curves, the calculations can only be done numerically using actual stress responses. In lieu of this more rigorous procedure, an approximation using the slope of the first branch of the $S–N$ curve (usually with $m = 3$) can be used, because most damage usually accumulates from this part.

A.16.7.3 Short-term stress range statistics

The GSR transfer function for a specific location in the structure and one wave direction is combined with the wave spectrum describing a sea state in the wave scatter diagram to calculate the GSR response spectrum for the sea state. The rms value and mean period of the GSR response are then computed by integrating the response spectrum and determining its moments. These parameters define the probability distribution function for GSR values and establish the short-term GSR statistics for the sea state. The Rayleigh distribution is an appropriate distribution function for narrow-banded processes and is normally applied. The Rice probability distribution function is valid for Gaussian random processes of any width and may be used as an alternative. The Rayleigh distribution returns stress peaks (or stress ranges) that are all positive. However, the Rice distribution includes a small fraction of the total number of stress peaks (or stress ranges) which are negative; for fatigue analysis purposes these negative stress ranges are removed from the distribution and ignored. See Figure A.16.7-1 for an outline of the overall procedure.

A.16.7.4 Long-term stress range statistics

No guidance is offered.

A.16.8 Determining the long-term stress range distribution by deterministic analysis

A.16.8.1 General

A flow chart for the procedure is shown in Figure A.16.8-1.

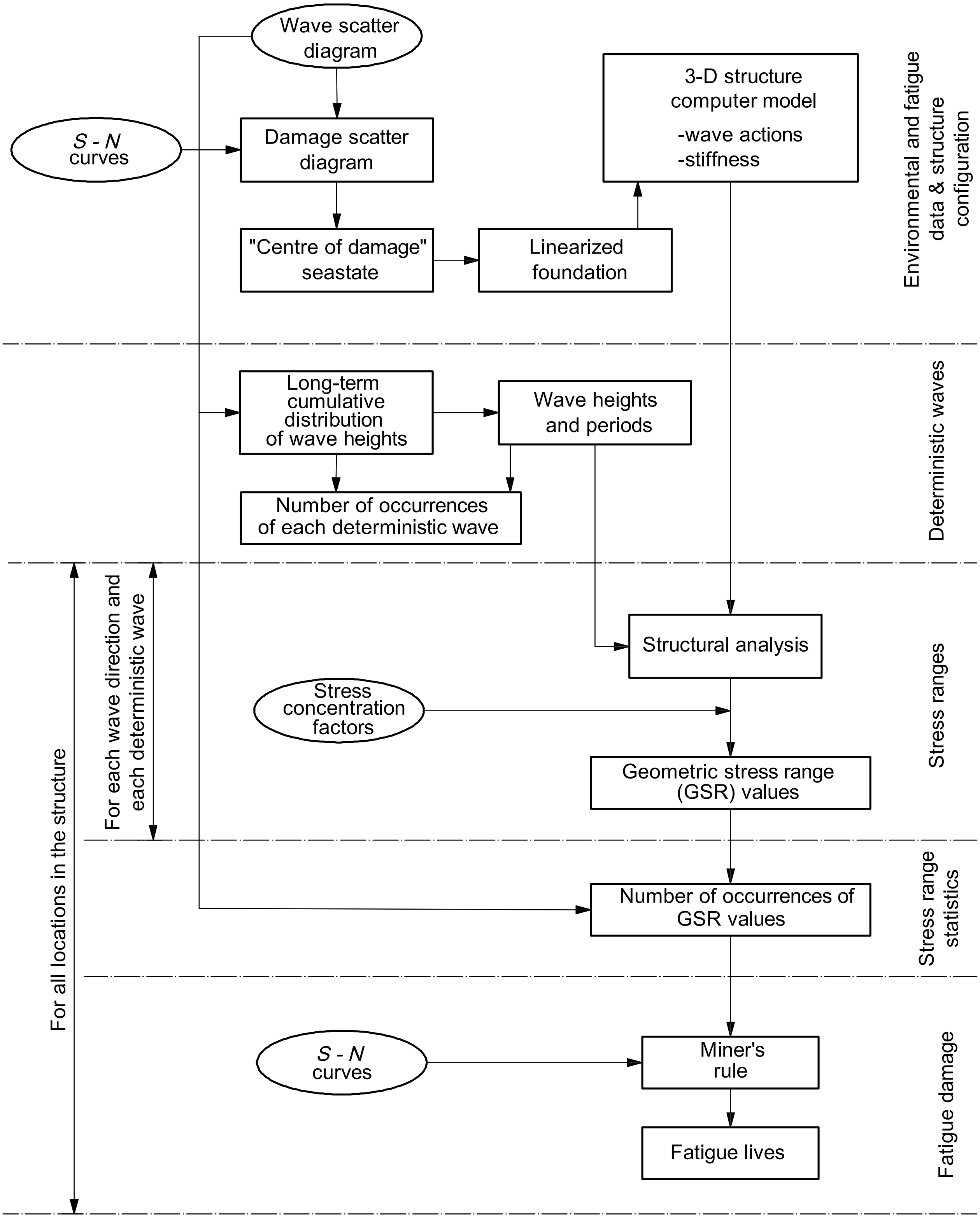


Figure A.16.8-1 — Flow chart of deterministic analysis procedure

As for a spectral analysis, for a deterministic analysis the GSR values are required for all locations in the structure that should be checked for fatigue. These are now calculated using a series of periodic (regular) waves.

For a given wave approach direction, a global structural analysis is performed using a periodic (regular) wave of suitable height and period. The analysis is performed in the time domain stepping the wave through the structure over a full cycle.

At each location in the structure and for each relevant stress component, the nominal stress and the associated phase with respect to the wave is determined. Using these stress components, the geometric stress $\sigma_{\mathrm{GS,A}}$ at point A is determined as described in 16.5. The range of $\sigma_{\mathrm{GS,A}}$ is the GSR at point A. This procedure is repeated for all points A and all locations in the structure.

The above calculations are next repeated for all wave periods for one particular wave approach direction, and subsequent to that for all wave approach directions.

The selection of the height and period of the regular waves is specific to the deterministic analysis and is described in 16.8.2 and 16.8.3.

A.16.8.2 Wave height selection

The long-term distribution of individual wave heights is calculated from the wave scatter diagram as described in 16.3.7. The number of waves occurring in each discrete wave height block is determined as shown in Figure A.16.3-3. The upper wave height of each block is normally used as the wave height of each regular wave for the global structural analysis.

A.16.8.3 Wave period selection

As the expected joint distribution of individual wave heights and periods for the offshore location under consideration is not normally available as routine information, the wave period selection should be established in consultation with an oceanographer.

The selected periods should also be verified against cancellation and addition frequencies in the shape of the transfer function of the total applied wave action; see 16.7.2.2. If any wave periods fall close to a relative minimum, the wave period should be adjusted to ensure a proper representation of the total wave energy. The lowest and highest, as well as intermediate wave frequencies, are less influential in the deterministic than in the spectral method, but should nonetheless be considered carefully. Considerations with respect to the natural frequencies of the structure are not relevant, as the deterministic method should not be used for dynamically responding structures.

A.16.8.4 Long-term stress range distribution

No guidance is offered.

A.16.9 Determining the long-term stress range distribution by approximate methods

A.16.9.1 The Weibull distribution of long-term stress ranges

The general three-parameter Weibull distribution of stress ranges is given by Equation (A.16.9-1):

$$P\left(S_{\mathrm{i}} > S_{\mathrm{i}}^{*}\right) = \exp\left[-\left(\frac{S_{\mathrm{i}}^{*} - A}{B}\right)^{C}\right] \tag{A.16.9-1}$$

where

S_{i} is the local geometric stress range;

$P(S_{\mathrm{i}} > S_{\mathrm{i}}^{*})$ is the probability that the stress range S_{i} exceeds the particular value S_{i}^{*};

A is the location parameter of the distribution ($S_i^* - A \geqslant 0$);

B is the scale parameter ($B > 0$);

C is the shape parameter ($C > 0$).

A Weibull distribution with the parameters (A, B, 1) is an exponential distribution, while a Weibull distribution with the parameters (0, 1, 2) is the normalized Rayleigh distribution.

For GSRs (S_i), the location parameter is zero ($A = 0$), while the probability $P(S_i > S_i^*)$ can further be expressed by Equation (A.16.9-2):

$$P\left(S_i > S_i^*\right) = \frac{N_{Si^*}}{N_T} \qquad \text{(A.16.9-2)}$$

where

N_{Si^*} is the number of times that S_i^* is exceeded during a period, T;

N_T is the total number of stress ranges occurring during period, T;

T is the duration considered, which for the in-place situation should be at least a year.

Substituting $A = 0$ and Equation (A.16.9-2) into Equation (A.16.9-1) and taking the natural logarithm results in:

$$\ln N_{Si^*} - \ln N_T = -\left(\frac{S_i^*}{B}\right)^C = -\left(B\right)^{-C}\left(S_i^*\right)^C \qquad \text{(A.16.9-3)}$$

All stress ranges exceed $S_i^* = 0$, so that $N_{Si^*=0} = N_T$, while there is only one maximum stress range, $S_{i,max}^*$, giving $N_{Si^*,max} = 1$. The scale parameter B can be determined by introducing $N_{Si^*,max} = 1$ into Equation (A.16.9-3):

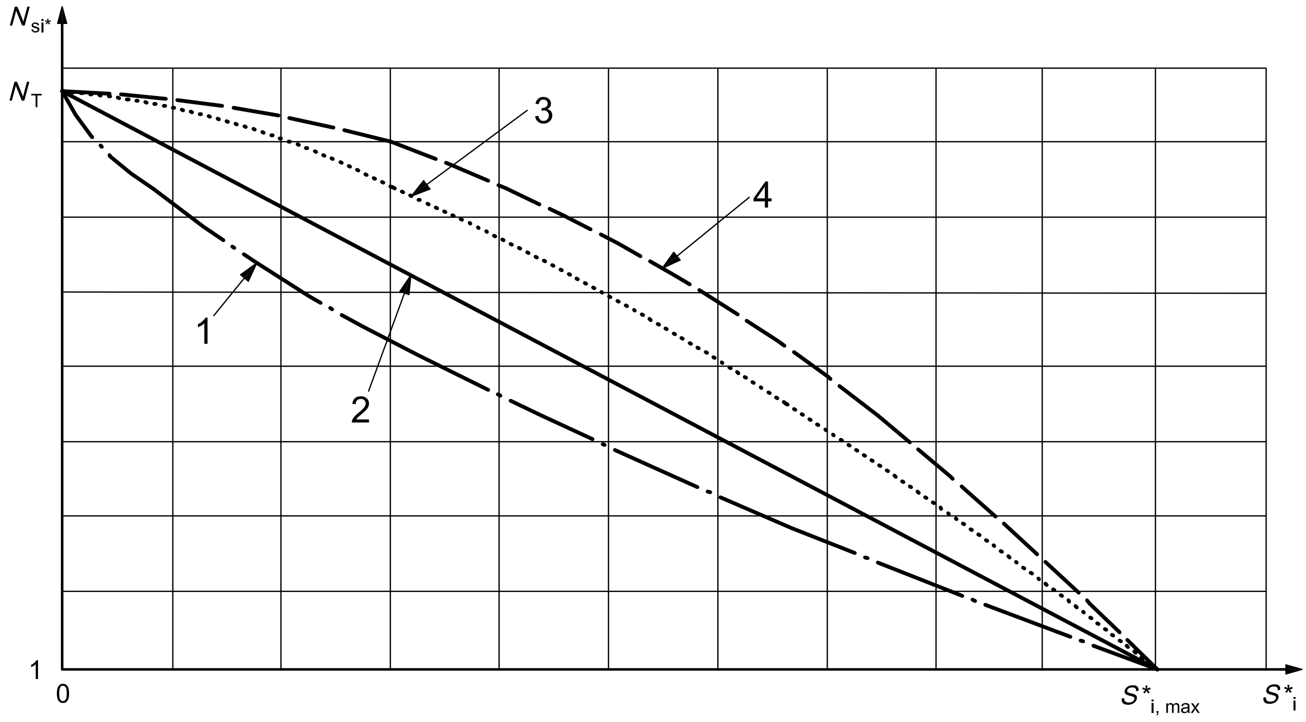
$$\left(B\right)^{-C} = \frac{\ln N_T}{\left(S_{i,max}^*\right)^C} \qquad \text{(A.16.9-4)}$$

Substituting this into Equation (A.16.9-3) provides the final equation of the Weibull distribution of local GSR as

$$\ln N_{Si^*} = \ln N_T - \frac{\ln N_T}{\left(S_{i,max}^*\right)^C}\left(S_i^*\right)^C \qquad \text{(A.16.9-5)}$$

If desired, the natural logarithm can be replaced by the logarithm on base 10 without any modification.

For a shape parameter, $C = 1$, this distribution plots as a straight line on a semi-logarithmic graph, with ln N_{Si^*} (or $\log_{10} N_{Si^*}$) on a logarithmic scale and S_i^* on a linear scale. For shape parameters, $C < 1$, the line is concave, while for shape parameters, $C > 1$, the line is convex. All lines pass through the two points (0, ln N_T) and ($S_{i,max}^*$, 1), see Figure A.16.9-1.



Key

1 Weibull distribution for $C = 0,7$

2 Weibull distribution for $C = 1,0$

3 Weibull distribution for $C = 1,5$

4 Weibull distribution for $C = 2,0$

S_i^* local geometric stress range (on linear scale)

$S_{i,max}^*$ maximum value of local geometric stress range

N_{Si^*} number of times that S_i^* is exceeded during period T (on logarithmic scale)

N_T total number of stress ranges occurring during period T

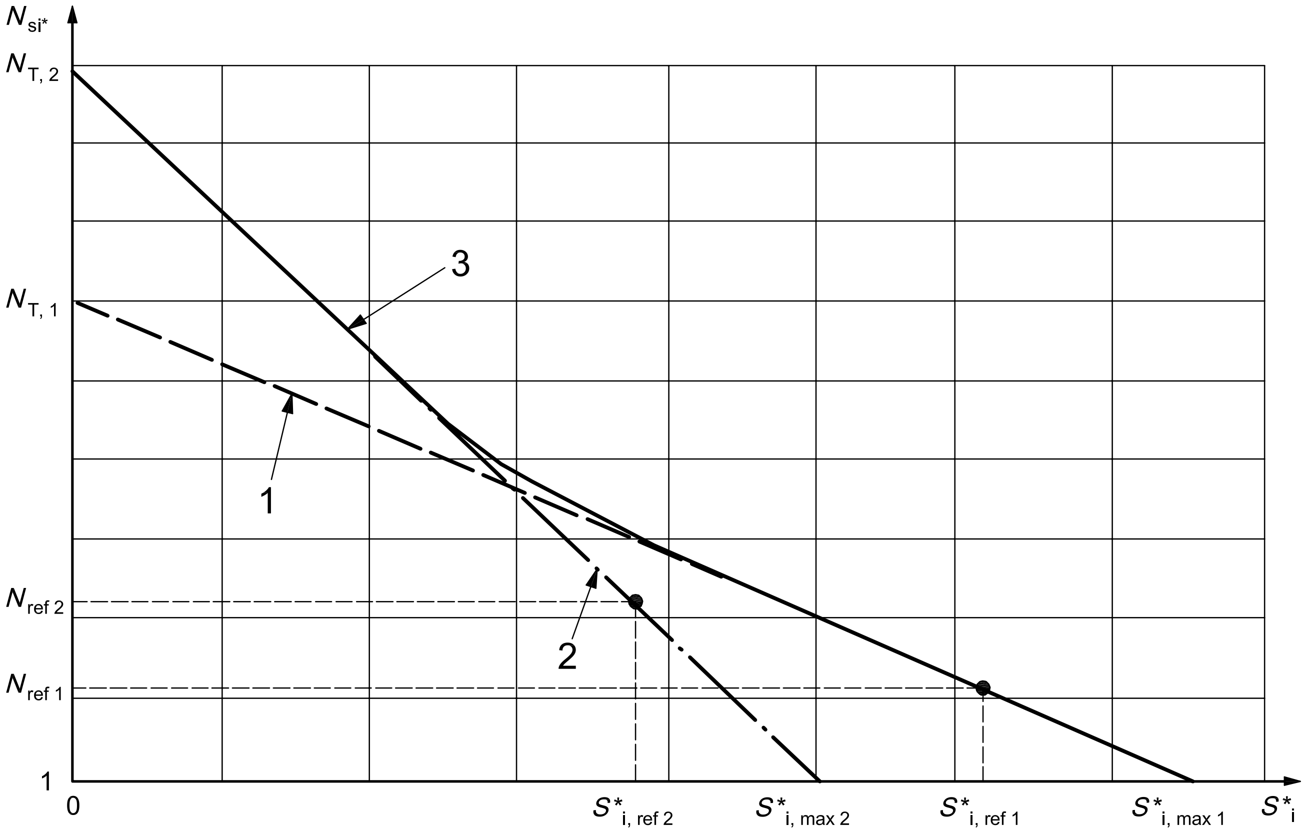
Figure A.16.9-1 — Weibull distributions

A.16.9.2 Determination of the distribution parameters

A.16.9.2.1 General

The Weibull distribution parameters of the transformed Equation (A.16.9-5) are N_T, $S_{i,max}^*$ and C. In general, these parameters can only be determined empirically by fitting Weibull distributions to measured or spectrally calculated long-term stress range distributions. For shape parameters, $C \neq 1$, this is the only possibility; there is no sound theoretical basis for estimating them otherwise.

However, experience has shown that in many cases a practical and reasonably accurate approximation is possible with a shape parameter, $C = 1$. For these cases, estimates of two points fully determine the resulting straight line graph on semi-logarithmic paper. One simple and convenient point that is usually chosen is (0, ln N_T), while the other point may be generally depicted as ($S_{i,ref}^*$, N_{ref}); see Figure A.16.9-2. If the long-term distribution of local GSRs is approximated by combining two Weibull distributions, the parameters of each distribution should be estimated separately; see Figure A.16.9-2.



Key

1 Weibull distribution for distribution 1

2 Weibull distribution for distribution 2

3 combined Weibull distributions

S_i^* local geometric stress range (on linear scale)

$S_{i,max}^*$ maximum value of local geometric stress range

N_{Si^*} number of times that S_i^* is exceeded during period T (on logarithmic scale)

N_T is the total number of stress ranges occurring during period T

Figure A.16.9-2 — Combination of Weibull distributions

A.16.9.2.2 Determination of the total number of stress range cycles

The total number of stress ranges, N_T, occurring during the period, T, may be derived from the total number of wave cycles occurring during T.

Where one exponential distribution adequately describes the long-term stress range distribution, T is normally chosen to be 1 year. The total number of waves per year, N_W, can be determined from the wave scatter diagram as discussed in A.16.3.7. N_T may next be related to N_W by a multiplication factor, R:

$$N_T = R \cdot N_W \quad \text{(A.16.9-6)}$$

Multiplication factor R reflects that shorter waves (high frequency waves, which form a large percentage of N_W) act over smaller depths below the surface than longer waves (low frequency waves, which form a relatively low percentage of N_W). In lieu of more pertinent information, the following values of R may be used as a guide:

R = 1,5 at and above 15 m below the still water surface for the fatigue analysis;

$R = 0{,}5$ at 30 m below the still water surface;

$R = 0{,}2$ at 60 m below the still water surface.

Between 30 m and 60 m below the still water surface, R may be determined by linear interpolation.

Where two exponential distributions should be combined to describe the long-term stress range distribution, T should normally be chosen to be the design service life (usually some 20 years). The first distribution then describes the stress ranges during the design service life that are due to wave conditions arising from normal atmospheric conditions, while the second distribution describes those that are due to the extraordinary wave conditions arising from cyclones. Reference [A.16.9-1] contains a typical example of such applications from the Gulf of Mexico, with $N_{T,1} = 10^9$ and $N_{T,2} = 10^6$.

A.16.9.2.3 Determination of point $S^*_{i,ref}$, N_{ref}

The local GSR values at all locations of interest are estimated using a single periodic (regular) wave in the same way as for the deterministic analysis procedure (see 16.8). A reference wave with height, H_{ref}, and period, T_{ref}, needs to be selected and is used as a fatigue design wave in the procedure. For cases governed by one exponential distribution two options readily present themselves:

a) the annual wave height occurring once a year with an associated wave period;

b) the most probable maximum wave at the centre of damage sea state [see A.16.7.2.3 and Equation (A.16.7-2)].

Other options reflecting and calibrated to local experience can be taken.

A stress analysis (see 16.4) is performed by stepping the reference wave through the structure, after which the GSR is determined (see 16.5) at all locations of interest in the structure. To ensure that no undue effects of partial cancellation occur due to phase differences between local wave actions on spatially distributed members, the wave length of the reference wave should be checked against the principal horizontal dimensions of the structure. The wave length should normally be more than four times the maximum dimension between the outer legs of the structure. The stress analysis is repeated for all wave directions relative to the structure. At each location of interest, the largest GSR calculated for any of the wave directions considered is $S^*_{i,ref}$. The corresponding N_{ref} is determined as T/T_{ref}.

Where two exponential distributions are involved, these steps should be executed for each of the two distributions separately.

NOTE The procedure for the Gulf of Mexico covered in Reference [A.16.9-1] is not based on estimating a second point of the distribution. Instead it defines long-term wave height distributions, one for regular wave conditions and one for hurricane conditions, then assumes a relationship between stress ranges and wave heights (regardless of wave period) and develops a closed-form formulation for fatigue damage.

A.16.9.2.4 Fatigue assessment

From the long-term distribution of local GSR values, the number of occurrences in discrete GSR blocks are determined as in Figure 16.7-1. These numbers are subsequently used in the fatigue damage calculation described in 16.12, as for the spectral and deterministic analysis procedures.

A.16.10 Geometrical stress ranges

A.16.10.1 General

No guidance is offered.

A.16.10.2 Stress concentration factors for tubular joints

A.16.10.2.1 General requirements for determination of stress concentration factor

A.16.10.2.1.1 Definition of stress concentration factor

The GSR concept has evolved as the most practical basis for fatigue design of tubular joints. This concept places many different structural geometries on a common basis, enabling them to be treated using a single S–N curve. The basis of this concept is to capture a stress (or strain) in the proximity of the weld toes, which characterizes the fatigue life of the joint, but excludes the very local microscopic effects such as the sharp notch, undercut and crack-like defects at the weld toe. These local weld notch effects are included in the S–N curve.

In this International Standard the terms *geometric stress* (GS) and *geometric stress range* (GSR) replace the previously used terms hot spot stress and hot spot stress range, except in the description of past (existing) practice. The stress concentration factor (SCF) can be defined as

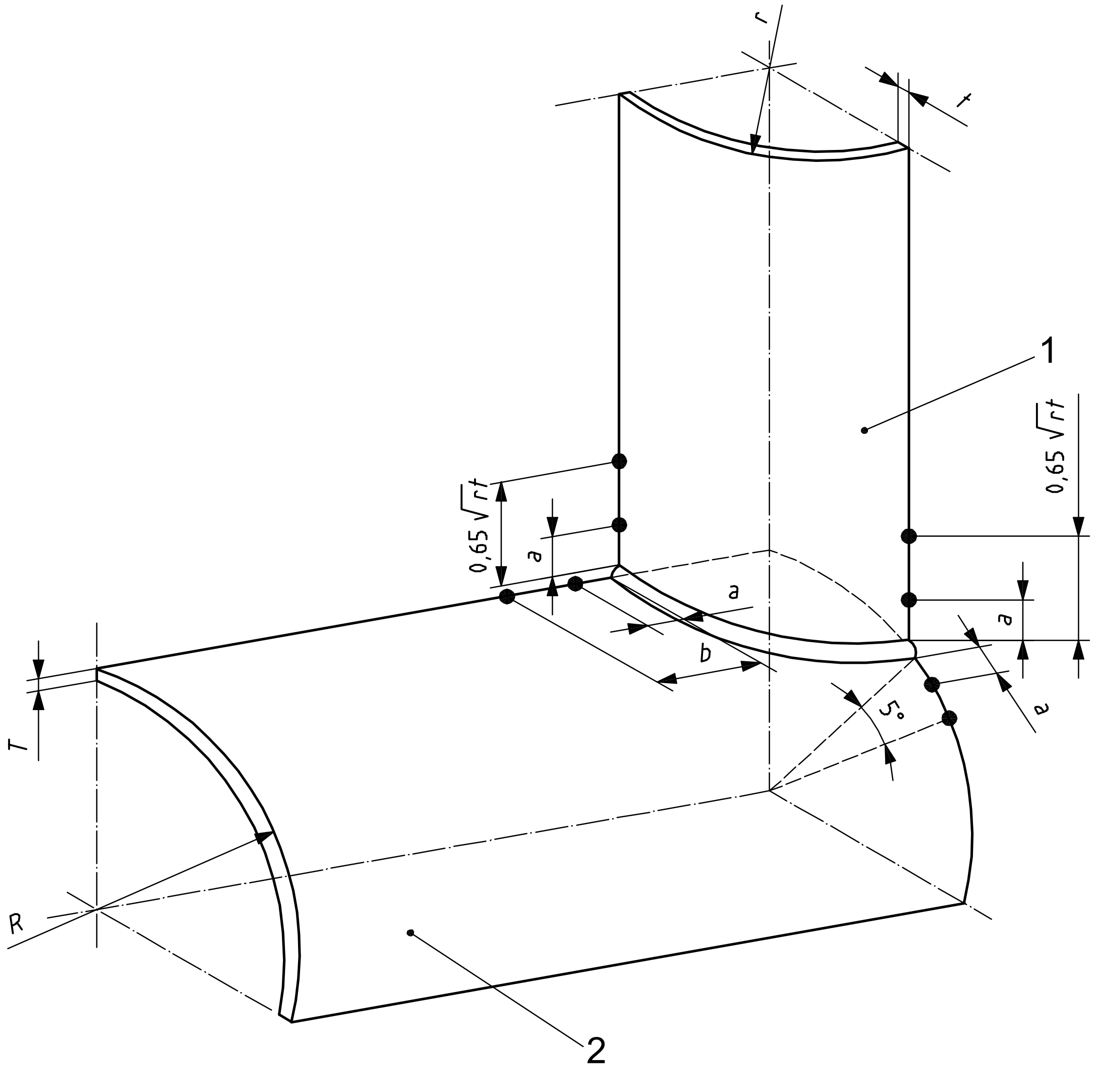
$$\text{SCF}=\frac{\text{the range of the GS at a particular location of the intersection weld (excluding notch effect)}}{\text{the range of the nominal brace stress}}$$

Consistency with the S–N curve is established by using the same method for estimating the GSR during the fatigue test as used when obtaining SCFs.

In US practice that is codified in API RP 2A [A.16.10-1] and AWS D1.1 [A.16.10-2] the geometric stress or strain is defined as the total range that would be measured by a strain gauge placed adjacent to the toe of the weld and oriented perpendicular to the weld so as to reflect the stress which will be amplified by the weld toe discontinuities [A.16.10-3]. Typical geometric strain gauges are centred within 6 mm to $0{,}1\sqrt{rt}$ from the weld toes with a gauge length of 3 mm. Here r and t refer to the outside radius and thickness of the member instrumented, whether chord or brace.

Within Europe, the geometric stress concept has been retained but applied differently. The local weld notch effect is excluded by using stress values just outside the weld notch region and extrapolating these (linearly) to the weld toe. The locations of the stress points used for extrapolation have been agreed upon by researchers in Europe, see Reference [A.16.10-4], and are defined in Figure A.16.10-1. The European definition is based on maximum principal stress, i.e. the stress components are extrapolated to the weld toes and then used in Mohr's Circle to establish the maximum principal stress at the toe. The stress normal to the weld toe, used in the US definition, is somewhat lower than this, but for the all-important saddle location the two are virtually identical.

The recommended S–N curves and SCF equations used in this International Standard are based on the European definition and are consistent.



Key

1 brace $a = 0{,}2\sqrt{r\,t}$, but $a \geqslant 4$ mm

2 chord $b = 0{,}4\sqrt[4]{r\,t\,R\,T}$

Figure A.16.10-1 — Distances for extrapolation to weld toes

A.16.10.2.1.2 Deriving SCFs

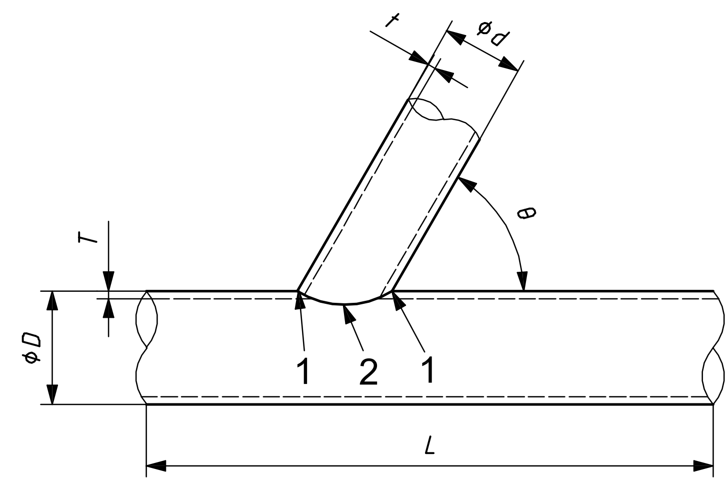
SCFs may be derived from FEA, model tests or empirical equations based on such methods. When deriving SCFs using FEA, solid (volume) elements should be used to represent the weld region (as opposed to thin shell elements). In models using volume elements, the SCFs can be derived by extrapolating stress components to the relevant weld toes and combining these to obtain the maximum principal stress and hence the SCF. The extrapolation direction should be normal to the weld toes.

If thin shell elements are used, the results should be interpreted carefully since no single method is guaranteed to provide consistently accurate stresses [A.16.10-5]. Extrapolation to the mid-surface intersection generally, but not consistently, overpredicts SCFs, whereas extrapolation to the notional weld toes would generally underpredict SCFs. In place of extrapolation, it is possible to directly use the nodal average stresses at the mid-surface intersection. This will generally overpredict stresses, especially on the brace side. This last method is expected to be more sensitive to the local mesh density than the extrapolation methods.

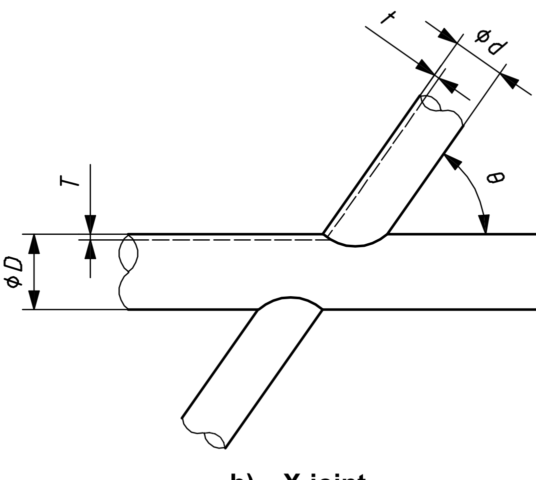
When deriving SCFs from model tests, care should be taken to cover all potential hot spot locations with strain gauges. Furthermore, it should be recognized that the strain concentration factor is not identical to the SCF but is related to it via the transverse strains and Poisson's ratio. If the chord length in the joint tested is less than about six diameters (chord length parameter $\alpha < 12$), the SCFs should possibly be corrected using the Efthymiou short-chord correction factors [A.16.10-6]. The same correction can be needed in FEA if $\alpha < 12$.

A.16.10.2.1.3 Joint classification

For the purpose of SCF evaluation, tubular joints are usually classified into joint types of T/Y-, X-, K-, or KT-joints, as defined in Figure A.16.10-2. The geometric parameters needed to define each joint type, e.g. chord diameter (D), chord thickness (T), brace diameter (d), brace thickness (t), etc. are shown in the figure. Appropriate non-dimensional, geometric parameters, β, γ, τ, α, θ and ζ, are also defined in Figure A.16.10-2 for each joint type.

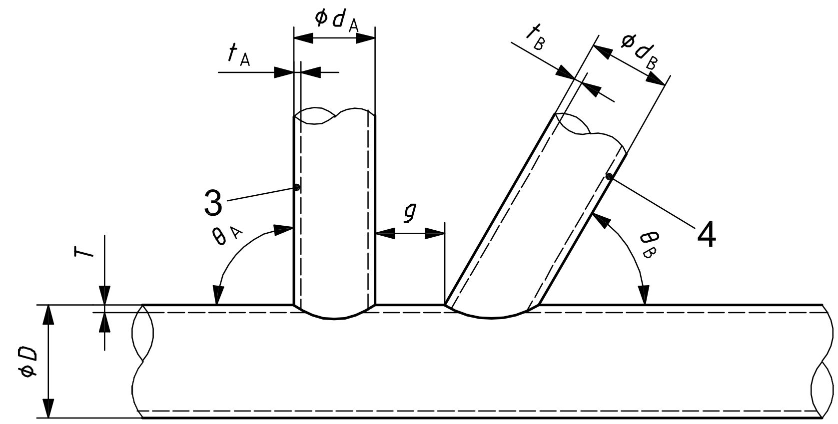


a) T- or Y-joint

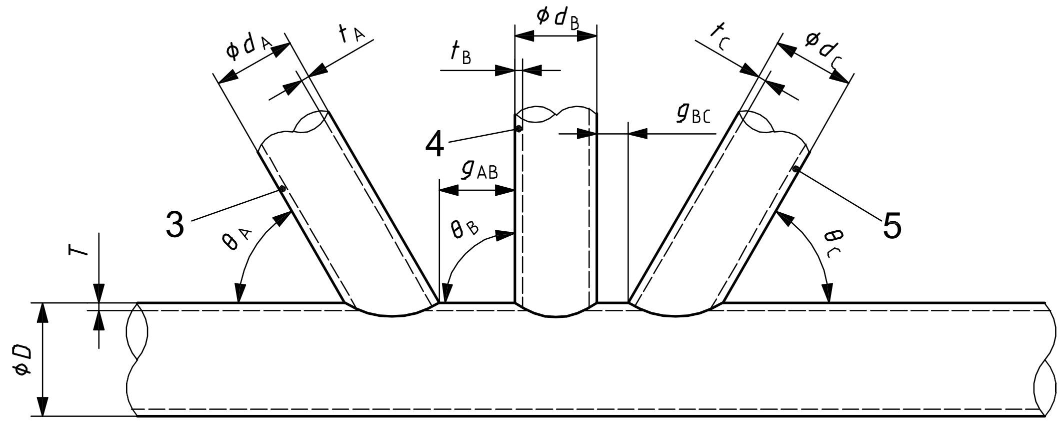


b) X-joint

Figure A.16.10-2 — Joint classification and geometrical parameters for SCFs (*continued*)



c) K-joint



d) KT-joint

Key

1	crown	$\beta = d/D$	$\beta_A = d_A/D$	$\beta_B = d_B/D$	$\beta_C = d_C/D$
2	saddle	$\tau = t/T$	$\tau_A = t_A/T$	$\tau_B = t_B/T$	$\tau_C = t_C/T$
3	brace A	$\zeta = g/D$	$\zeta_{AB} = g_{AB}/D$	$\zeta_{BC} = g_{BC}/D$	
4	brace B	$\gamma = D/2\,T$			
5	brace C	$\alpha = L/2\,D$			

Figure A.16.10-2 — Joint classification and geometrical parameters for SCFs (*continued*)

For many joints it is sufficiently accurate to use a classification based on geometry only, following the rules given in Table A.16.10-1, or the assumptions on forces in brace members recommended below. Alternatively, a classification methodology can be used, including the more rational influence function approach which handles classification automatically on the basis of the actual load paths. This is discussed further in A.16.10.2.2.

Table A.16.10-1 — Classification table for a brace end

N_{near}	N_{far}	**Classification** [a]
1	0	T/Y
1	1	X
1	2	T/Y
1	3	T/Y
2	Any	K
3	Any	KT

N_{near} number of braces on same side of chord as reference brace (including reference brace)
N_{far} number of braces on far side of chord with respect to reference brace

[a] Non-planar braces are ignored.

In joint types where more than one brace is involved, i.e. in K-, KT- and X-joints, the relative magnitude and direction of the nominal brace forces (and moments) has a significant influence on SCFs. For instance, for a typical K-joint under opb, the SCF increases from 2 when the brace moments are exactly balanced, to approximately 6 when they are unbalanced.

The following recommendations are given with respect to categorization of tubular K-, KT- and X-joints based on force patterns in braces for fixed offshore structures; see also 14.2.4.

— **K- and KT-joints**

1) normal components of the axial brace forces are assumed to be balanced;

2) ipb moments on braces are assumed to be unbalanced;

3) opb moments on braces are assumed to be unbalanced.

— **X-joints**

1) axial forces on braces are assumed to be balanced;

2) SCFs in X-joints are not sensitive to the sign of the ipb moment;

3) opb moments on braces are assumed to be balanced.

These recommendations are based on the assumption that, in a well-braced structure, the response of primary joints is usually governed by axial forces; bending and shear due to frame action are of secondary importance. Hence the (normal components of the) axial forces in the braces at a joint (X-, K- or KT-joints) should be approximately balanced so that shear remains small. Ipb and opb moments can be significant when they are caused by direct wave action (as opposed to frame action) and for secondary braces. Hence ipb and opb moments in K- and KT-joints tend to be unbalanced; while opb moments in X-joints tend to be balanced for the same reason.

A.16.10.2.1.4 Evaluation of GSRs

The key GSR locations at the tubular joint intersection are termed saddle and crown (see Figure A.16.10-2). A minimum of eight stress range locations need to be considered around each chord/brace intersection weld in order to adequately cover all relevant locations. These are: the chord sides at two crown positions, the brace sides at two crown positions, the chord sides at two saddle positions and the brace sides at two saddle positions. The GSRs for the chord and the brace side of the weld are determined from the geometric stresses given by Equation (A.16.10-1):

$$\begin{aligned}\sigma_{\mathrm{GS,s}}(t) &= C_{\mathrm{ax,s}}\ \sigma_{\mathrm{ax}}(t) \pm C_{\mathrm{opb,s}}\ \sigma_{\mathrm{opb}}(t) \\ \sigma_{\mathrm{GS,c}}(t) &= C_{\mathrm{ax,c}}\ \sigma_{\mathrm{ax}}(t) \pm C_{\mathrm{ipb,c}}\ \sigma_{\mathrm{ipb}}(t) + \sigma_{\mathrm{C,c}}(t)\end{aligned} \tag{A.16.10-1}$$

where

σ_{GS} is the geometric stress on the chord or the brace side of the weld between chord and brace;

σ_{ax} is the nominal axial stress in the brace (or stub);

σ_{ipb} is the nominal in-plane bending stress in the brace (or stub);

σ_{opb} is the nominal out-of-plane bending stress in the brace (or stub);

$\sigma_{\mathrm{C,c}}$ is the nominal stress in the chord (or chord can) at the crown position, see A.16.10.2.1.5;

C_{ax} is the stress concentration factor for axial brace stress;

C_{ipb} is the stress concentration factor for in-pane bending stresses in the brace;

C_{opb} is the stress concentration factor for out-of-plane bending stresses in the brace;

t is time;

s is the subscript denoting the saddle position;

c is subscript denoting the crown position.

The GSR, denoted by S, is the maximum minus the minimum stress during one full stress cycle. Equations (A.16.10-1) apply to both the brace side and the chord side of the weld at the intersection of brace and chord. Note that the nominal chord stress ($\sigma_{\mathrm{C,c}}$) only affects the chord side of the weld at the crown position and should be deleted from Equation (A.16.10-1) when the geometric stress for the brace is determined.

Since the nominal brace stresses, σ_{ax}, σ_{opb} and σ_{ipb}, are functions of wave position (i.e. time t), it follows that in combining the contributions from the various stresses, the phase differences between them are automatically taken into account.

For some joints and certain individual brace force patterns, the point of highest stress can lie at a location between the saddle and the crown. Examples include balanced axial brace forces in K-joints where the hot spot generally lies between the saddle and the crown toe. For ipb the hot spot might not be precisely at the crown, but could lie within a sector of $\pm 30°$ from the crown depending on the γ- and β- values. The recommended SCF equations capture these higher SCFs even though they are referred to as occurring notionally at the crown or the saddle for simplicity.

For combined axial forces and bending moments, it is possible for the maximum GSR to occur at a location between the saddle and crown, even when the individual hot spots occur at the saddle or crown. These cases occur when opb and ipb contributions are comparable in terms of GSR and are in-phase, and when, additionally, the axial contributions are small or relatively constant around the intersection. For such cases, Equation (A.16.10-1) can underpredict the maximum stress range. To overcome this, the GSR around the entire joint intersection can be estimated using location dependent SCFs and a generalized form of Equation (A.16.5-1):

$$S(\chi) = C_{\mathrm{ax,C}}(\chi) \cdot \sigma_{\mathrm{ax}}(t) \pm C_{\mathrm{ipb,\,C}}(\chi) \cdot \sigma_{\mathrm{ipb}}(t) \pm C_{\mathrm{opb,\,C}}(\chi) \cdot \sigma_{\mathrm{opb}}(t)$$
$$S(\chi) = C_{\mathrm{ax,B}}(\chi) \cdot \sigma_{\mathrm{ax}}(t) \pm C_{\mathrm{ipb,\,B}}(\chi) \cdot \sigma_{\mathrm{ipb}}(t) \pm C_{\mathrm{opb,\,B}}(\chi) \cdot \sigma_{\mathrm{opb}}(t) \quad \text{(A.16.10-2)}$$

where, additionally,

$S(\chi)$ is the GSR at the location around the chord to brace intersection defined by angle χ;

$C_{\mathrm{ax,C}}(\chi)$ describes the variation of the SCF due to axial brace force on the chord side around the chord to brace intersection defined by angle χ;

$C_{\mathrm{ipb,C}}(\chi)$ and $C_{\mathrm{opb,C}}(\chi)$ similarly describe the variation of the SCFs due to in-plane and out-of-plane bending around the chord to brace intersection defined by angle χ, respectively.

SCFs with subscript B instead of C describe the same variables for the brace side of the chord to brace intersection. The SCF distribution functions around the circumference can be obtained from parametric expressions given in Reference [A.16.10-7].

A.16.10.2.1.5 Effect of nominal chord stress $\sigma_{\mathrm{C,c}}(t)$

Nominal variable stresses in the chord member also contribute to stress ranges for fatigue damage accumulation. Their contribution is usually small because (unlike brace forces) chord forces do not cause any significant local distortion of the chord walls. Hence any stress-raising effects are minimal. The effect of nominal variable stresses in the chord member can be covered by including the stress ($\sigma_{\mathrm{ax,\,C}}$) due to axial force in the chord can member, combined with an axial SCF of 1,25, i.e. $\sigma_{\mathrm{C,c}}(t) = 1{,}25\ \sigma_{\mathrm{ax,\,C}}(t)$, while accounting for sign and phase differences with other effects due to brace force patterns in Equation (A.16.10-1). The contribution, $\sigma_{\mathrm{C,c}}(t)$, is applied at the chord crown location only; contributions at other locations, i.e. at the chord saddle and at the brace side, are considerably smaller and may be neglected.

A.16.10.2.2 Unstiffened tubular joints

A.16.10.2.2.1 Review of SCF equations

Several sets of parametric equations have been derived for estimating SCFs in tubular joints, see References [A.16.10-5], [A.16.10-6] and [A.16.10-8] to [A.16.10-11].

The performance of the various sets of SCF equations in terms of accuracy, degree of conservatism and range of applicability has been assessed in a number of studies, notably in a study by the Edison Welding Institute (EWI) funded by API, see Reference [A.16.10-12], and a study by Lloyd's Register funded by HSE, see Reference [A.16.10-7].

The main conclusion from the EWI study was that the Efthymiou equations [A.16.10-6] and the Lloyd's design equations [A.16.10-7] have considerable advantages in consistency and coverage in comparison with other available equations. When discussing the Lloyd's SCF equations it is important to clarify that two modern sets of Lloyd's SCF equations exist, namely:

— mean SCF equations created from the database of acrylic test results that were available in 1988;

— design SCF equations defined as mean-plus-one standard deviation from the same database.

When assessed by EWI against the latest SCF database, the Lloyd's mean SCF equations are found to generally underpredict SCFs and fail the HSE assessment criteria. The mean SCF equations are not recommended for design.

A second conclusion from the EWI study was that the option of mixing-and-matching equations from different sets would lead to inconsistencies and is not recommended. An inconsistency of this type is already present in the Lloyd's design SCF equations, due to the variable partial factor used, but it is not very significant.

For the α-Kellogg equations which are given in Reference [A.16.10-13], the EWI study concluded that they are generally more conservative than both the Efthymiou and the Lloyd's design SCF equations. Perhaps the most significant weakness of the α-Kellogg equations is that the predicted SCFs for all joint types are independent of β. This is clearly not the case as evidenced from test data and FEA results. Furthermore, the equations imply that chord SCFs are proportional to $\sqrt{\gamma}$ as opposed to observations which indicate that they increase linearly with γ. One advantage of the α - Kellogg equations is their simplicity.

In the comparison studies by Lloyd's Register, the Efthymiou SCF equations were found to provide a good fit to the screened SCF database, with a bias of about 10 % to 25 % on the conservative side [A.16.10-7]. They generally pass the HSE criteria for goodness-of-fit and conservatism. For the important case of K-joints under balanced axial forces, the Efthymiou equations did not pass the HSE criteria. A closer examination of this specific case revealed that these equations are satisfactory for both the chord and the brace side. For the chord side in particular, the Efthymiou equations provide the best fit to the database (COV = 19 %) and have a bias of 19 % on the conservative side. The second best equation (Lloyd’s) has a COV of 21 % and a bias of 41 % on the conservative side. The HSE criteria were deliberately designed to favour those equations that overpredict SCFs and to penalize underpredictions. This is the key reason why the Efthymiou equations for K-joints marginally failed the HSE criteria, even though they provide a good fit and also err on the conservative side.

The Lloyd’s design SCF equations generally pass the HSE criteria, except for T/Y-joints under axial force and ipb on the brace crown side. The reason for this is that there seems to be a systematic difference between the acrylic results and steel results for T/Y-joints at the brace crown.

Use of the Efthymiou SCF equations is recommended because this set of equations is considered to offer either the best option or a very good option for all joint types and types of brace forces and is the only set which covers overlapped K- and KT-joints.

Mix-and-match between different sets of equations is not recommended. The Efthymiou equations are also recommended in the 22nd edition of API RP2A WSD (see Reference [A.16.10-14]). The Efthymiou equations are given in Tables A.16.10-2 to A.16.10-5 and briefly discussed in A.16.10.2.2.2.

A.16.10.2.2.2 The Efthymiou equations

a) Overview

The Efthymiou equations cover SCFs in unstiffened T/Y-, X-, K- and KT-joints under all relevant brace force conditions. Overlapped K- and KT-joints are also covered. These expressions are based on extensive FEA using the PMBSHELL program. The program uses thick shell elements for modelling the chord and braces and 3-D brick elements for the welds. The weld profiles are as per AWS D1.1. This modelling enables direct extrapolation of stresses to the weld toes. The modelling and extrapolation removes the need for corrections, such as those attributed to Marshall [A.16.10-3], which were aimed at the brace side SCFs derived from thin shell FEA. Inclusion of the weld profiles with appropriate cut-back for high diameter ratios ensures realistic behaviour when modelling $\beta = 1{,}0$ joints.

b) T/Y-joints

For T/Y-joints under axial brace forces (see Table A.16.10-2) the SCFs are significantly influenced by the chord length and the fixity conditions at the ends of the chord. Beam bending of the chord influences primarily the crown SCFs, while for short chords ($\alpha < 12$), the shell distortion at the chord to brace intersection and hence the SCFs are affected by the fixity at the ends.

For beam bending the chord end fixity is defined by a parameter C, which is analogous to the effective length factor for buckling and has the range of 0,5–1,0. When $C = 0{,}5$, the ends are fully fixed and the equations degenerate to those of the fixed case. When $C = 1{,}0$ the chord ends are pinned. There are instances where beam bending of the chord is limited (see below). In such cases the chord length should be taken to be small (e.g. $\alpha = 12$ or less).

For joints with short chords ($\alpha < 12$), the correction factors, F_1 or F_2, in the Efthymiou equations should be applied when the ends of the chord are radially restrained. Factor F_1 is applicable when the ovalization at the chord ends is completely suppressed, for instance by a diaphragm, or if the ends are welded onto another stiff member.

c) X-joints

In X-joints under axial brace forces (see Table A.16.10-3) the SCFs at the saddles tend to dominate for all values of β, including the common case of $\beta = 1{,}0$. In joints with short chords the correction factors, F_1 or F_2, in the Efthymiou equations may be used, provided that ovalization of the ends is restrained to some extent. If the ends are completely free, the saddle SCFs should be increased. An approximate way of achieving this is to increase them by the ratio $1/F_2$.

The chord crown SCF (C_{X2}) under axial forces is derived from the corresponding SCF (C_{T2}) for T/Y-joints by suppressing beam bending of the chord, i.e. setting α to zero. For the brace crown SCF (C_{X4}), a better fit to the PMBSHELL database was obtained not by setting α to zero but by deleting completely the second term in the corresponding SCF (C_{T4}) for T/Y-joints.

d) K-joints

K-joints with a gap greater than one chord diameter ($\zeta > 1$) under balanced axial brace forces (Table A.16.10-4), should be classified as Y-joints for the purpose of SCF evaluation. The chord length parameter, α, should be set to 12 to reflect the fact that beam bending of the chord is limited. For opb when $\zeta > 1$, the SCF equations degenerate to the Y-joint equations and hence it is not necessary to re-classify these joints.

e) KT-joints

For KT-joints under balanced axial brace forces (see Table A.16.10-5) the SCFs on a diagonal brace are evaluated by considering the axial brace force to be balanced by the other diagonal brace, i.e. ignoring the central brace and hence degenerating the joint to a K-joint. For the central brace it is generally sufficient to consider that its axial force is balanced by one of the diagonal braces. If the diagonal braces are identical but the gaps differ, the maximum of the two gaps should be used, to be conservative. If the diagonal braces are not identical, then the central brace should be successively paired with each of the diagonal braces and the maximum resulting SCFs selected.

f) Validity ranges

The validity ranges for the Efthymiou equations are as follows:

$$\begin{array}{ccccc} 0{,}2 & \leqslant & \beta & \leqslant & 1{,}0 \\ 0{,}2 & \leqslant & \tau & \leqslant & 1{,}0 \\ 8 & \leqslant & \gamma & \leqslant & 32 \\ 4 & \leqslant & \alpha & \leqslant & 40 \\ 20^\circ & \leqslant & \theta & \leqslant & 90^\circ \\ \dfrac{-0{,}6\beta}{\sin\theta} & \leqslant & \zeta & \leqslant & 1{,}0 \end{array} \tag{A.16.10-3}$$

For cases where one or more parameters fall outside this range, the following procedure should be adopted:

1) evaluate SCFs using the actual values of geometric parameters;

2) evaluate SCFs using the limit values of geometric parameters;

3) use the maximum of 1) or 2) above in the fatigue analysis.

Table A.16.10-2 — Equations for SCFs in T/Y-joints

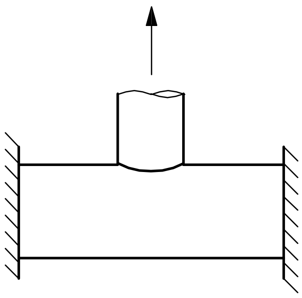
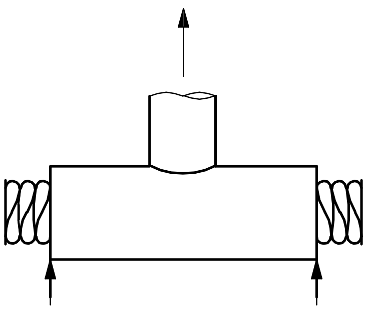
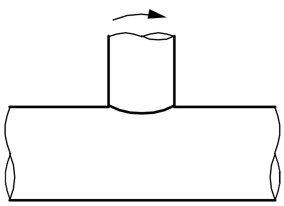
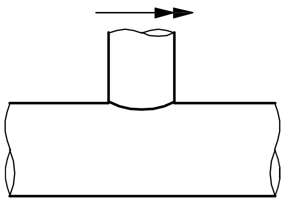
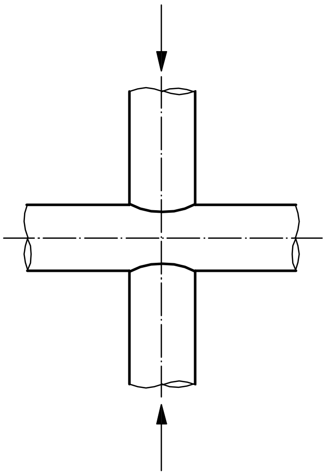
Type of brace force and fixity conditions	SCF equation
Axial brace force, chord ends fixed 	Chord saddle: $C_{CS} = F_1\, C_{T1}$ $\quad C_{T1} = \gamma\, \tau^{1,1}\left[1,11 - 3\left(\beta - 0,52\right)^2\right]\sin^{1,6}\theta$ Chord crown: $C_{CC} = C_{T2}$ $\quad C_{T2} = \gamma^{0,2}\tau\left[2,65 + 5\left(\beta - 0,65\right)^2\right] + \tau\, \beta\left(0,25\alpha - 3\right)\sin\theta$ Brace saddle: $C_{BS} = F_1\, C_{T3}$ $\quad C_{T3} = 1,3 + \gamma\, \tau^{0,52}\alpha^{0,1}\left[0,187 - 1,25\beta^{1,1}\left(\beta - 0,96\right)\right]\sin^{(2,7-0,01\alpha)}\theta$ Brace crown: $C_{BC} = C_{T4}$ $\quad C_{T4} = 3 + \gamma^{1,2}\left[0,12\exp\left(-4\beta\right) + 0,011\beta^2 - 0,045\right] + \beta\, \tau\left(0,1\alpha - 1,2\right)$
Axial brace force, general chord fixity 	Chord saddle: $C_{CS} = F_2\, C_{T5}$ $\quad C_{T5} = C_{T1} + C_1\left(0,8\alpha - 6\right)\tau\beta^2\left(1 - \beta^2\right)^{0,5}\sin^2 2\theta$ Chord crown: $C_{CC} = C_{T6}$ $\quad C_{T6} = \gamma^{0,2}\tau\left[2,65 + 5\left(\beta - 0,65\right)^2\right] + \tau\, \beta\left(C_2\alpha - 3\right)\sin\theta$ Brace saddle: $C_{BS} = F_2\, C_{T3}$ $\quad$ See above Brace crown: $C_{BC} = C_{T7}$ $\quad C_{T7} = 3 + \gamma^{1,2}\left[0,12\exp\left(-4\beta\right) + 0,011\beta^2 - 0,045\right] + \beta\tau\left(C_3\alpha - 1,2\right)$
In-plane bending (ipb) 	Chord crown: $C_{CC} = C_{T8}$ $\quad C_{T8} = 1,45\beta\tau^{0,85}\gamma^{(1-0,68\beta)}\sin^{0,7}\theta$ Brace crown: $C_{BC} = C_{T9}$ $\quad C_{T9} = 1 + 0,65\beta\tau^{0,4}\gamma^{(1,09-0,77\beta)}\sin^{(0,06\gamma-1,16)}\theta$
Out-of-plane bending (opb) 	Chord saddle: $C_{CS} = F_3\, C_{T10}$ $\quad C_{T10} = \gamma\, \tau\, \beta\left(1,7 - 1,05\beta^3\right)\sin^{1,6}\theta$ Brace saddle: $C_{BS} = F_3\, C_{T11}$ $\quad C_{T11} = \tau^{-0,54}\gamma^{-0,05}\left(0,99 - 0,47\beta + 0,08\beta^4\right)\times C_{T10}$

Table A.16.10-2 (*continued*)

Type of brace force and fixity conditions	SCF equation
	Short chord correction factors (α < 12): $F_1 = 1 - \left(0{,}83\beta - 0{,}56\beta^2 - 0{,}02\right)\gamma^{0{,}23}\exp\left(-0{,}21\gamma^{-1{,}16}\alpha^{2{,}5}\right)$ $F_2 = 1 - \left(1{,}43\beta - 0{,}97\beta^2 - 0{,}03\right)\gamma^{0{,}04}\exp\left(-0{,}71\gamma^{-1{,}38}\alpha^{2{,}5}\right)$ $F_3 = 1 - 0{,}55\beta^{1{,}8}\gamma^{0{,}16}\exp\left(-0{,}49\gamma^{-0{,}89}\alpha^{1{,}8}\right)$ where $\exp(x) = e^x$
	Chord-end fixity parameter, C: $0{,}5 \leqslant C \leqslant 1{,}0$ (Typically $C = 0{,}7$) $C_1 = 2(C - 0{,}5)$ $C_2 = C/2$ $C_3 = C/5$

Table A.16.10-3 — Equations for SCFs in X-joints

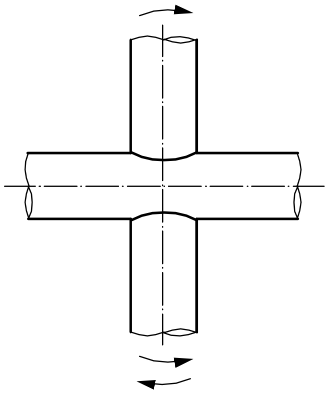
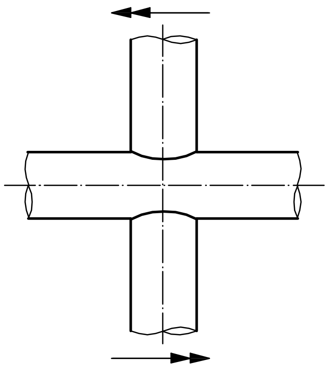
Type of brace force	SCF equation
Axial force (balanced) 	Chord saddle: $C_{CS} = C_{X1}$ $\quad C_{X1} = 3{,}87\gamma\tau\beta\left(1{,}10 - \beta^{1{,}8}\right)\sin^{1{,}7}\theta$ Chord crown: $C_{CC} = C_{X2}$ $\quad C_{X2} = \gamma^{0{,}2}\tau\left[2{,}65 + 5(\beta - 0{,}65)^2\right] - 3\,\tau\beta\sin\theta$ Brace saddle: $C_{BS} = C_{X3}$ $\quad C_{X3} = 1 + 1{,}9\gamma\tau^{0{,}5}\beta^{0{,}9}\left(1{,}09 - \beta^{1{,}7}\right)\sin^{2{,}5}\theta$ Brace crown: $C_{BC} = C_{X4}$ $\quad C_{X4} = 3 + \gamma^{1{,}2}\left[0{,}12\exp(-4\beta) + 0{,}011\beta^2 - 0{,}045\right]$

In joints with short chords, $\alpha < 12$, which have stiffened ends, both the chord saddle and the brace saddle SCF may be reduced by multiplying them by the short chord factor, F_1 or F_2. Factor F_1 can be used for stiff end reinforcements preventing ovalization as well as rotation of the chord wall, while factor F_2 can be used for end reinforcements partially preventing ovalization only.

If the chord ends are completely free, both the chord saddle and the brace saddle SCF can increase significantly. An approximation can be obtained by increasing them by the ratio 1,0/F_2 (see A.16.10.2.2.2), but FEA is recommended.

F_1 and F_2 are given in Table A.16.10-2.

Table A.16.10-3 (*continued*)

Type of brace force	SCF equation
In-plane bending (ipb) (balanced or unbalanced) 	Chord crown: $C_{\mathrm{CC}} = C_{\mathrm{T8}}$ see Table A.16.10-2 Brace crown: $C_{\mathrm{BC}} = C_{\mathrm{T9}}$ see Table A.16.10-2
Out-of-plane bending (opb) (balanced) 	Chord saddle: $C_{\mathrm{CS}} = C_{\mathrm{X5}}$ $\quad C_{\mathrm{X5}} = \gamma\ \tau\ \beta\left(1{,}56 - 1{,}34\beta^4\right)\sin^{1{,}6}\theta$ Brace saddle: $C_{\mathrm{BS}} = C_{\mathrm{X6}}$ $\quad C_{\mathrm{X6}} = \tau^{-0{,}54}\gamma^{-0{,}05}\left(0{,}99 - 0{,}47\beta + 0{,}08\beta^4\right)\times C_{\mathrm{X5}}$

In joints with short chords, $\alpha < 12$, which have ends stiffened with a diaphragm or ring stiffener, both the chord saddle and the brace saddle SCF may be reduced by multiplying them by the short chord factor F_3; see Table A.16.10-2.

If the chord ends are completely free, saddle SCFs can increase significantly. An approximation can be obtained by increasing them by the ratio $1{,}0/F_3$ (see A.16.10.2.2.2), but FEA is recommended.

Table A.16.10-4 — Equations for SCFs in gap/overlap K-joints

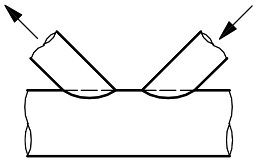
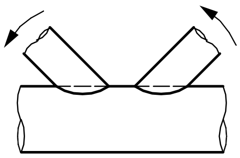
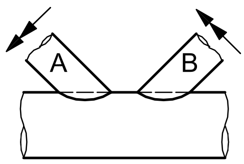
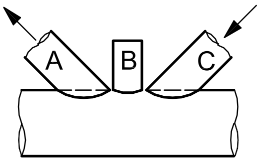
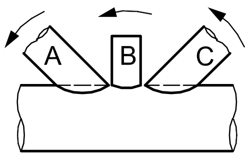
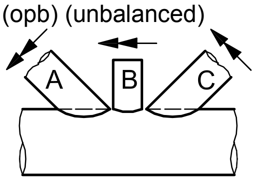
Type of brace force	SCF equation
Axial forces (balanced) 	Chord: $C_C = C_{K1}$ $\quad C_{K1} = \left[\tau^{0,9}\gamma^{0,5}\left(0,67-\beta^2+1,16\beta\right)\sin\theta\right]\left(\frac{\sin\theta_{max}}{\sin\theta_{min}}\right)^{0,30} \times \left(\frac{\beta_{max}}{\beta_{min}}\right)^{0,30}$ $\times\left[1,64+0,29\beta^{-0,38}\arctan\left(8\zeta\right)\right]$ Brace: $C_B = C_{K2}$ $\quad C_{K2} = 1+C_{K1}\left(1,97-1,57\beta^{0,25}\right)\left(\tau^{-0,14}\sin^{0,7}\theta\right)+$ $\left[K\,\beta^{1,5}\gamma^{0,5}\tau^{-1,22}\sin^{1,8}\left(\theta_{max}+\theta_{min}\right)\right]\times\left[0,131-0,084\arctan\left(14\zeta+4,2\beta\right)\right]$ where $K = 0$ for gap joints; $K = 1$ for the through brace; $K = 0,5$ for the overlapping brace; the arctangents are evaluated in radians; τ, β, θ and the nominal stress relate to the brace being considered.
In-plane bending (ipb) (unbalanced) 	Chord crown, non-overlapping joint or overlap $\leqslant$ 30 % of contact length: $C_{CC} = C_{T8}$ see Table A.16.10-2 Chord crown, overlap $>$ 30 % of contact length: $C_{CC} = 1,2\,C_{T8}$ see Table A.16.10-2 Brace crown, non-overlapping joint: $C_{BC} = C_{T9}$ see Table A.16.10-2 Brace crown, overlapping joint: $C_{BC} = C_{T9}\times\left(0,9+0,4\beta\right)$
Out-of-plane bending (opb) (unbalanced)[a] 	Chord saddle adjacent to brace A: $C_{CS} = F_4\,C_{K4}$ $\quad C_{K4} = C_{T10,A}\left[1-0,08\left(\beta_B\,\gamma\right)^{0,5}\exp\left(-0,8x\right)\right]$ $+C_{T10,B}\left[1-0,08\left(\beta_A\,\gamma\right)^{0,5}\exp\left(-0,8x\right)\right]\times\left[2,05\beta_{max}^{0,5}\exp\left(-1,3x\right)\right]$ where $x = 1+\frac{\zeta\sin\theta_A}{\beta_A}$ $C_{T10,A}$ and $C_{T10,B}$ (see Table A.16.10-2) are calculated with the parameters for braces A and B respectively. Brace saddle adjacent to brace A: $C_{BS} = F_4\,C_{K5}$ $\quad C_{K5} = \tau^{-0,54}\gamma^{-0,05}\left(0,99-0,47\beta+0,08\beta^4\right)\times C_{K4}$
	Short chord correction factor (α < 12) $F_4 = 1-1,07\beta^{1,88}\exp\left[-0,16\gamma^{-1,06}\alpha^{2,4}\right]$ where $\exp\left(x\right) = e^x$
[a] The designation of braces A and B is not geometry dependent. It is nominated by the user.	

Table A.16.10-5 — Equations for SCFs in KT-joints

Type of brace force	SCF equation
Axial force (balanced) 	Chord: $C_{\mathrm{C}} = C_{\mathrm{K1}}$ see Table A.16.10-4 Brace: $C_{\mathrm{B}} = C_{\mathrm{K2}}$ see Table A.16.10-4 where $\zeta = \zeta_{\mathrm{AB}} + \zeta_{\mathrm{BC}} + \beta_{\mathrm{B}}$ for the diagonal braces A and C; ζ = maximum of ζ_{AB} and ζ_{BC} for the central brace B.
In-plane bending (ipb) 	Chord crown: $C_{\mathrm{CC}} = C_{\mathrm{T8}}$ see Table A.16.10-2 Brace crown: $C_{\mathrm{BC}} = C_{\mathrm{T9}}$ see Table A.16.10-2
Out-of-plane bending (opb) (unbalanced) 	Chord saddle at diagonal brace A: $C_{\mathrm{CS}} = C_{\mathrm{KT1}} \quad C_{\mathrm{KT1}} = C_{\mathrm{T10,A}}\left[1-0{,}08(\beta_{\mathrm{B}}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_{\mathrm{AB}})\right]\times\left[1-0{,}08(\beta_{\mathrm{C}}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_{\mathrm{AC}})\right] + C_{\mathrm{T10,B}}\left[1-0{,}08(\beta_{\mathrm{A}}\gamma)^{0{,}5}\exp(-0{,}8\,x_{\mathrm{AB}})\right]\times\left[2{,}05\beta_{\max}^{0{,}5}\exp(-1{,}3\,x_{\mathrm{AB}})\right] + C_{\mathrm{T10,C}}\left[1-0{,}08(\beta_{\mathrm{A}}\gamma)^{0{,}5}\exp(-0{,}8\,x_{\mathrm{AC}})\right]\times\left[2{,}05\beta_{\max}^{0{,}5}\exp(-1{,}3\,x_{\mathrm{AC}})\right]$ where $x_{\mathrm{AB}} = 1 + \dfrac{\zeta_{\mathrm{AB}}\sin\theta_{\mathrm{A}}}{\beta_{\mathrm{A}}}$ $x_{\mathrm{AC}} = 1 + \dfrac{(\zeta_{\mathrm{AB}}+\zeta_{\mathrm{BC}}+\beta_{\mathrm{B}})\sin\theta_{\mathrm{A}}}{\beta_{\mathrm{A}}}$ Chord saddle at central brace B: $C_{\mathrm{CS}} = C_{\mathrm{KT2}} \quad C_{\mathrm{KT2}} = C_{\mathrm{T10,B}}\left[1-0{,}08(\beta_{\mathrm{A}}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_{\mathrm{AB}})\right]^{(\beta_{\mathrm{A}}/\beta_{\mathrm{B}})^2} \times\left[1-0{,}08(\beta_{\mathrm{C}}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_{\mathrm{BC}})\right]^{(\beta_{\mathrm{C}}/\beta_{\mathrm{B}})^2} + C_{\mathrm{T10,A}}\left[1-0{,}08(\beta_{\mathrm{B}}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_{\mathrm{AB}})\right]\times\left[2{,}05\beta_{\max}^{0{,}5}\exp(-1{,}3\,x_{\mathrm{AB}})\right] + C_{\mathrm{T10,C}}\left[1-0{,}08(\beta_{\mathrm{B}}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_{\mathrm{BC}})\right]\times\left[2{,}05\beta_{\max}^{0{,}5}\exp(-1{,}3\,x_{\mathrm{BC}})\right]$ where $x_{\mathrm{AB}} = 1 + \dfrac{\zeta_{\mathrm{AB}}\sin\theta_{\mathrm{B}}}{\beta_{\mathrm{B}}}$ $x_{\mathrm{BC}} = 1 + \dfrac{\zeta_{\mathrm{BC}}\sin\theta_{\mathrm{B}}}{\beta_{\mathrm{B}}}$ Brace saddle: $C_{\mathrm{BS}} = C_{\mathrm{KTB}} \quad C_{\mathrm{KTB}} = \tau^{-0{,}54}\gamma^{-0{,}05}\left(0{,}99-0{,}47\beta+0{,}08\beta^4\right)\times C_{\mathrm{CS}}$

A.16.10.2.2.3 Guidance on reducing the SCF

Increasing the chord wall thickness is an effective way of reducing stress concentrations. For T/Y-and X-joints, a doubling of the chord wall thickness reduces the saddle SCFs by a factor of 4; crown SCFs are also reduced considerably.

Increasing the brace wall thickness (by introducing brace stubs) is NOT an effective way of reducing stress concentrations. For many joint configurations, the introduction of thicker brace stubs has little or no effect on the GSR on the chord side. It actually leads to an increase in the SCF, which is counteracted by a corresponding decrease in the nominal brace stub stress, and hence there is no significant change in the fatigue damage calculation. The use of thicker brace stubs tends to reduce the GSR on the brace side, but this reduction is small and usually not worthwhile in terms of the controlling stress value.

It is also relevant to note that the introduction of thicker brace stubs has little or no effect on the static strength of joints. It does, of course, increase the strength of the brace end and hence can be an attractive design detail for a brace that carries substantial transverse actions (e.g. direct wave action or actions due to accidental ship impact on braces in the splash zone). In general, however, the use of thicker brace stubs is an inefficient use of steel and should be avoided.

The use of thicker brace stubs further introduces additional stress concentrations at the brace to stub closure welds in nodal constructions because of the thickness transition. This can create additional problems and the use of thicker brace stubs should be avoided.

A.16.10.2.2.4 Influence functions

The concept of influence functions as a generalization of the SCF method of evaluating GSRs is described in References [A.16.10-6], [A.16.10-11] and [A.16.10-15]. This method is more accurate than the SCF approach because it can handle generalized forces and moments on the braces forming the joint, as opposed to an approach that is based on individual planes and joint classification.

An influence function is an expression for the GSR at a certain location around the chord-to-brace intersection arising from a nominal stress of unit magnitude acting on any brace of the joint. The GSR at a given location can be obtained by multiplying each influence function with its respective nominal stress and superimposing the contributions from all braces of the joint, including the brace under consideration.

The influence function algorithm is consistent with the SCF approach in the sense that it will lead to identical results for a joint that is subjected to forces and classed in the manner that is assumed by the SCF approach. For instance, it leads to identical results for a K-joint when the axial forces are exactly balanced and the opb moments are exactly unbalanced.

In addition to being more accurate than the SCF approach, the influence function concept obviates the need to classify joints and hence is more convenient to use. An additional advantage is that it has been extended [A.16.10-6], [A.16.10-11] to handle multiplanar joints for the important case of axial brace forces.

A disadvantage of the influence function algorithm is that it is less transparent than the direct SCF approach.

For complex joints of particular interest, specific influence coefficients and geometric stresses can be accurately established by developing a detailed local FEA model of the joint and incorporating this model into the overall fatigue analysis (frame) model of the structure [A.16.10-16]. The advantage of this approach is that it captures all brace and chord forces and moments and their phase differences and all geometric stress concentration effects, including multiplanar effects. However, introducing local joint flexibility can also have an effect on global stresses; therefore, when taking this approach, all joints should in principle have flexibility modelled.

A.16.10.2.2.5 Tubular joints welded from one side

Single-sided welding is used as the principal method for connecting braces to chords in tubular joints for offshore structures in many areas of the world. Single-sided welding presupposes that a fatigue crack always initiates at the weld toe. However, if the SCF at the internal weld root of a tubular joint is relatively high compared to that at the external weld toe (e.g. the internal SCF is 70 % of the external SCF), then the crack can initiate at the internal weld root, due to more onerous S–N curves that are relevant for the root detail than for the external weld toe. This is particularly important when weld improvement techniques are employed.

The appropriate S–N curve from the other joints (OJ) group (see Table 16.11-1) should be used for fatigue analysis of the weld root. For further information, see References [A.16.10-17] and [A.16.10-18].

A.16.10.2.2.6 Tubular thickness transitions

Thickness transitions introduce additional stress concentrations at the connection weld and hence potential fatigue problems. The highest stresses occur on that side of the transition that is not flush. Possible fatigue problems are therefore made worse if the transition is made internally because the highest stresses then occur at the weld root, which has the highest potential for welding defects. Furthermore, this location cannot easily be inspected. Where thickness transitions are necessary, they should therefore be external (i.e. internally flush connections).

A.16.10.2.3 Internally ring stiffened tubular joints

The Lloyd's equations for ring-stiffened joints are given in Reference [A.16.10-19]. The following points should be noted regarding the equations:

a) the derived SCF ratios for the brace to chord intersection and the SCFs for the ring edge are mean values, although the degree of scatter and proposed design factors are given;

b) short chord effects should be taken into account where relevant;

c) for joints with diameter ratios $\beta \geqslant 0{,}8$, the effect of stiffening is uncertain, it may even increase the SCF;

d) the maximum of the saddle and crown values should be applied around the whole brace to chord intersection;

e) the minimum SCF for the brace side under axial and opb moments should be taken as 2,0, whereas a minimum value of 1,5 is recommended for all other locations.

The following additional observations can be made about the use of ring stiffeners in general:

— thin shell FEA should be avoided for calculating the SCF if the maximum stress is expected to be near the brace to ring crossing point, and special consideration should be given to this crossing point in the fatigue analysis;

— ring stiffeners have a marked effect on the circumferential stress in the chord, but have little or no effect on the longitudinal stress;

— ring stiffeners outside the brace footprint have little effect on the SCF but can be beneficial for static strength.

Failures in the ring inner edge or brace to ring interface occur internally and will probably only be detected after through-thickness cracking, at which time the majority of the fatigue life will have been expended. These areas should therefore be considered as non-inspectable, unless sophisticated inspection methods are used.

A.16.10.2.4 Grouted tubular joints

Grouted joints have either the chord completely filled with grout (single skin grouted joints) or the annulus between the chord and an inner member filled with grout (double skin grouted joints). The SCF of a grouted joint depends on the stress history. The SCF is smaller when the bond between the chord and the grout is unbroken. Due to the grout, the tensile and compressive SCFs can be different. Only the highest value should be used in the fatigue analysis.

Grouted joints may be treated as simple joints, except that for the brace and the chord saddle points the chord thickness in the γ- term for saddle SCF calculations may be substituted with an equivalent chord wall thickness given by Equation (A.16.10-4):

$$T_e = (5\,D + 134\,T)/144 \qquad \text{(A.16.10-4)}$$

where

D is the chord outside diameter;

T is the chord wall thickness.

The formulation in Equation (A.16.10-4) has been derived on the basis of engineering mechanics.

Joints with high β- or low γ- ratios gain little benefit from grouting. Although fully substantiated evidence is not available, the benefits of grouting should be neglected for joints with $\beta > 0{,}9$ or $\gamma \leqslant 12{,}0$, unless documented otherwise. A minimum SCF value of 1,5 is recommended for all locations.

A.16.10.2.5 Cast joints

It is recommended that FEA be used to determine the magnitude and location of the maximum stress range in castings sensitive to fatigue. The FEA model should use volume elements at the critical areas and properly model the shape of the joint. Consideration should be given to stresses at the inside of the castings. The brace to casting circumferential butt weld, which is designed to the appropriate S–N curve from the OJ group (see Table 16.11-1), can be the most critical location for fatigue.

In the absence of fatigue data for castings tested in sea water under normal cathodic protection conditions, a minimum SCF of at least two should be applied.

A.16.10.3 Geometric stress ranges at other fatigue-sensitive locations

In any welded connection there are several locations at which a fatigue crack can develop, e.g. in the parent metal at the weld toe, at the weld ends and at the weld root (in the case of the weld throat of load carrying fillet or partial penetration butt welds). Each location should be classified separately.

Constructional details for non-welded material and welded connections with their corresponding joint classifications are given in, firstly, Table A.16.10-6, which affords an overview, and in Tables A.16.10-7 to A.16.10-12.

Table A.16.10-6 — Joint classification tabulation

Table	Type	Constructional detail
A.16.10-7	Type 1	Material free from welding
A.16.10-8	Type 2	Continuous welds essentially parallel to the direction of applied stress
A.16.10-9	Type 3	Transverse groove welds in plates and tubulars (essentially perpendicular to the direction of applied stress)
A.16.10-10	Type 4	Welded attachments on the surface or edge of a stressed member
A.16.10-11	Type 5	Load-carrying fillet and T-butt welds
A.16.10-12	Type 6	Details in welded girders and tubulars

The types and associated joint classification have been established on the basis of stresses generally along (within 45°) the direction indicated in the tables by the arrow for the potential crack on the surface of the parent metal or, in the case of weld throat cracking, on the shear stress calculated in the weld throat. The classification is based on a minimum quality level for assembly of joint and weld, see, for example, Reference [A.16.10-2].

For details that are not expressly classified, the following minimum classification class should be used, unless a higher class can be justified from published experimental work, or by specific tests:

— W_1 for load carrying fillet or partial penetration weld metal;

— F_2 for other cases.

The appropriate stress range indicated in Tables A.16.10-7 to A.16.10-11 and used as the GSR is the maximum principal stress range adjacent to the detail under consideration, except for the throat of load carrying fillet or partial penetration welds, for which it is the shear stress range calculated on the minimum throat area. The stress range should be amplified to reflect the effect of any stress concentration in the vicinity of the joint that is not characteristic of the detail itself. For example, effects of holes, cut-outs, re-entrant corners, joint eccentricity and misalignment during fabrication that is greater than allowable should be taken into account, either in the structural model used to calculate stress range or by appropriate stress concentration factors. SCFs for these typical details may be obtained from References [A.16.10-20], [A.16.10-21] and other published literature.

Table A.16.10-7 — Type 1: material free from welding

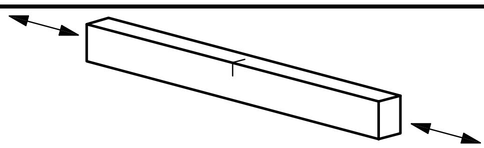
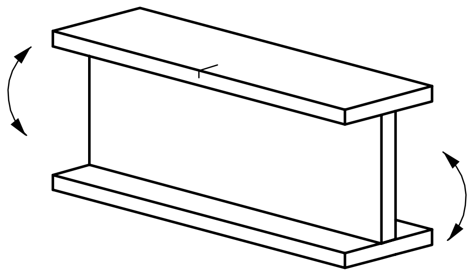
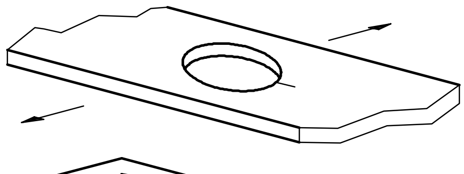
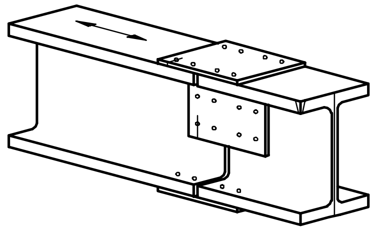
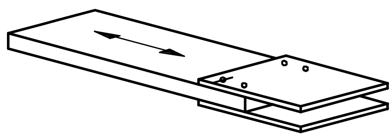
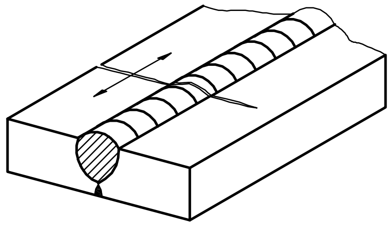
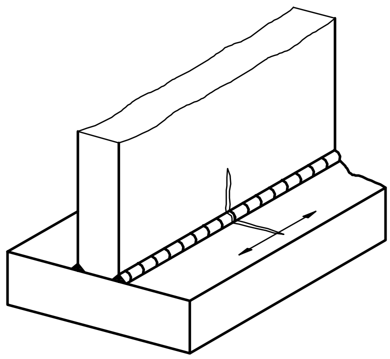
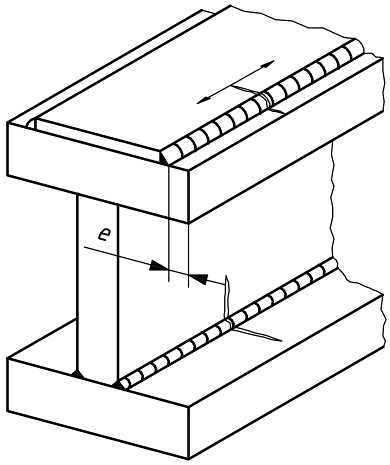
Type number and description, explanatory comments	Class	Example, including failure mode
1.1 Plain steel a) In the as-rolled condition or ground smooth or machined after cutting.	B	 
d) With edges flame cut. No repair by weld refill	C	
1.2 Bolted connection Unsupported one-sided connections should be avoided or the effect of eccentricities taken into account in the calculation of stresses. Stresses to be calculated in the gross section for friction grip connections or in the net section for all other connections.	C	 
NOTE In plain steel, fatigue cracks initiate at the surface, usually either at surface irregularities or at corners of the cross-section. In welded construction, fatigue failure will rarely occur in a region of plain material since the fatigue resistance of the welded joints will usually be much lower. In steel with holes or other stress concentrations arising from the shape of the member, failure will usually initiate at the stress concentration. In this case, the stress range should include an SCF.		

Table A.16.10-8 — Type 2: continuous welds essentially parallel to the direction of applied stress

Type number and description, explanatory comments	Class	Example, including failure mode
2.1 Complete or partial joint penetration groove weld. Parent or weld metal in members, without attachments, built up of plates or sections and joined by continuous welds.	B	
a) Complete joint penetration groove welds with the weld overfill dressed flush with the surface and machine-finished in the direction of stress, and with the weld proved free from significant defects by non-destructive testing.	B	
b) Groove or fillet welds with the welds made from both sides by an automatic submerged or open arc process and with no stop-start positions within the length. If an accidental stop-start occurs remedial action should be taken so that the finished weld has a similar surface and root profile to that intended.	C	
c) As b) weld from one side.	D	
d) As b) but with the weld containing stop-start positions within the length. For the ends of flange cover plates, see joint type 6.4.	D	
e) As b) but with manual welding.	D	e is the distance from weld toe to edge of flange, $e > 10$ mm
2.2 Discontinuous weld: refer to type 6.5.		

Remarks on potential modes of failure

With the excess weld metal dressed flush, fatigue cracks would be expected to initiate at weld defect locations. In the as-welded condition, cracks might initiate at stop-start positions or, if these are not present, at weld surface ripples.

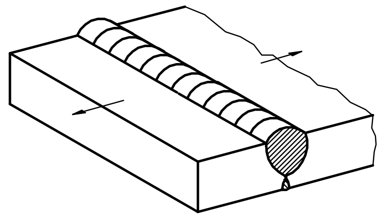
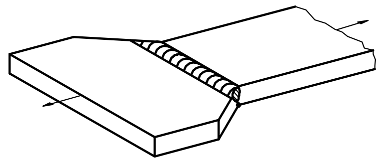
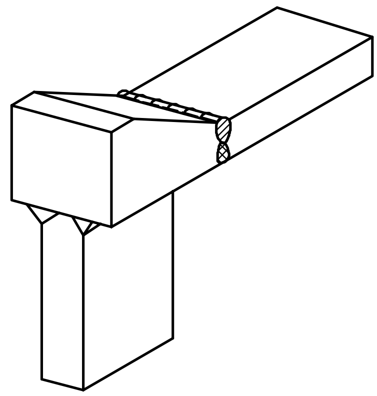
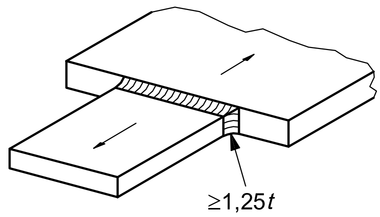
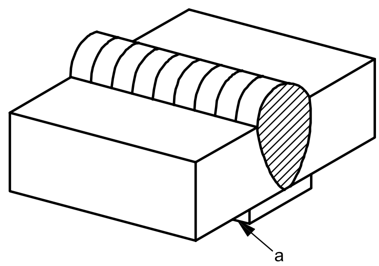
General comments

1) Backing strips: if backing strips are used in making these joints: a) they should be continuous, and b), if they are attached by welding, those welds should also comply with the relevant type requirements (note particularly that tack welds, unless subsequently ground out or covered by a continuous weld, would reduce the joint to joint type 6.5).

2) Edge distance: an edge distance criterion exists to limit the possibility of local stress concentrations occurring at unwelded edges as a result, for example, of undercut, weld spatter, or accidental over weave in manual fillet welding (see also notes on joint type 4). Although an edge distance can be specified only for the "width" direction of an element, it is equally important to ensure that no accidental undercutting occurs on the unwelded corners of, for example, cover plates or box girder flanges. If it does occur, it should subsequently be ground smooth.

3) Weld ends: weld ends are classified as type 4 or 5.

Table A.16.10-9 — Type 3: transverse groove welds in plates and tubulars (i.e. essentially perpendicular to the direction of applied stress)

Type number and description, explanatory comments	Class	Example
3.1 Parent and weld metal at complete penetration butt joints welded from both sides. Changes in thickness should be machined to a smooth transition not steeper than 1 in 4. Changes of less than 1,15 times the thinner member thickness may be accommodated in the weld profile without machining.		
a) With the weld cap ground flush with the surface and the weld proved to be free from significant defects by NDT.	C	
b) With the weld made, either manually or by an automatic process other than submerged arc, provided all runs, including repairs, are made in the down-hand position.	D	
c) In general, welds made by the submerged arc process, or in positions other than down-hand, tend to have a poor reinforcement shape, from the point of view of fatigue resistance. Hence, such welds are downgraded from c) to d).		
d) Welds made other than in a), b) or c). In both c) and d) the corners of the cross-section of the stressed component at the weld toes should be dressed to a smooth profile.	E	
e) Weld between plates of unequal width, with the weld ends ground to a radius not less than 1,25 times the thickness, t. Step changes in width can often be avoided by the use of shaped transition plates. For this detail the stress concentration has been taken into account in the joint classification.	F_2	
3.2 Parent and weld metal at complete joint penetration butt joints made from one side on a permanent backing strip. If the backing strip is fillet welded or tack welded to the member, the joint should be assessed using joint type 4 c).	F	
3.3 Parent and weld metal at full penetration weld made from one side without permanent backing strip. All circumferential welds in tubulars of less than 650 mm diameter should be considered single-sided.	F_2	a No tack weld.

Remarks on potential modes of failure

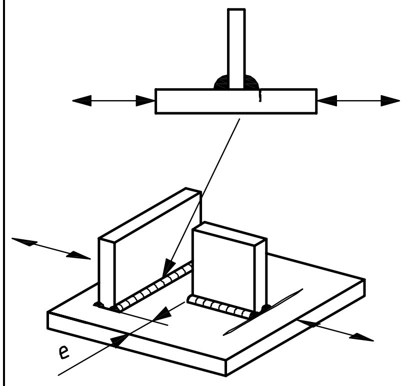
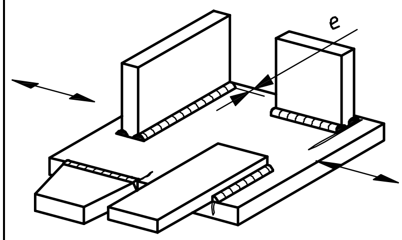
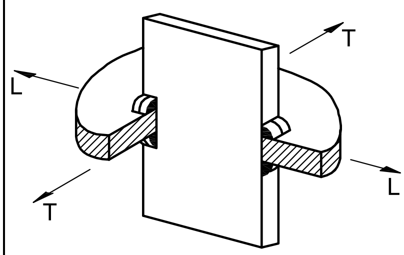
With the weld ends machined flush with the plate edges, fatigue cracks in the as-welded condition normally initiate at the weld toe, so that the fatigue resistance depends largely upon the shape of the weld overfill. If this is dressed flush, the stress concentration caused by it is removed and failure is then associated with weld defects. In welds made on a permanent backing strip, fatigue cracks initiate at the weld metal to strip junction, and in partial joint penetration groove welds at the weld root.

Welds made entirely from one side, without a permanent backing, require care to be taken in the making of the root bead in order to ensure a satisfactory profile.

General comments

The stress range considered should be that at the actual crack location. It should include SCF's arising from any thickness change, or due to the overall form of the joint (e.g. butt joint at the end of a conical transition piece).

Table A.16.10-10 — Type 4: welded attachments on the surface or edge of a stressed member

Type number and description, explanatory comments	Class	Examples including failure mode
4. Welded attachments Parent metal of the stressed member adjacent to toes or ends of groove or fillet welded attachments (both stressed and unstressed attachments), regardless of the orientation of the weld to the direction of applied stress, and whether or not the welds are continuous around the attachment. Groove welds should have an additional reinforcing fillet so as to provide a similar toe profile to that which would exist in a fillet welded joint. Parent metal of the stressed member at the toe of a full penetration weld connecting the stressed member to another member slotted through it.		 e = edge distance
a) With attachment length (parallel to the direction of the applied stress) $\leqslant$ 150 mm and with edge distance $\geqslant$ 10 mm. This classification includes ring stiffeners on tubulars away from nodes.	F	 e = edge distance
b) With attachment length (parallel to the direction of the applied stress) $>$ 150 mm and with edge distance $\geqslant$ 10 mm. The decrease in fatigue resistance with increasing attachment length is caused by more force being transferred into the longer gusset, giving an increase in stress concentration.	F_2	
c) Weld within 10 mm of the edges or corners of a stressed member. The classification applies to all sizes of attachment. It would therefore include, for example, the junction of two flanges at right angles. In such situations, a low fatigue classification can often be avoided by the use of a transition plate [see also joint type 3.1 e)].	G	

Remarks on potential modes of failure

When the weld is parallel to the direction of the applied stress, fatigue cracks normally initiate at the weld ends, but when it is transverse to the direction of stressing, they usually initiate at the weld toe; for attachments involving a single, as contrasted to a double, weld, cracks can also initiate at the weld root. The cracks then propagate into the stressed member. When the welds are on or adjacent to the edge of the stressed member, the stress concentration is increased and the fatigue resistance is reduced; this is the reason for specifying an "edge distance" in some of these joints (see also remark on edge distance in joint type 2).

General comments

1) The stress range considered should be the nominal stress range in the member on which the attachment is welded.

2) When the attachment is subjected to significant forces, affecting the stress flow in the member, another joint type has to be used, e.g. type 6.6 for gusseted connections.

Table A.16.10-11 — Type 5: load-carrying fillet and T-butt welds

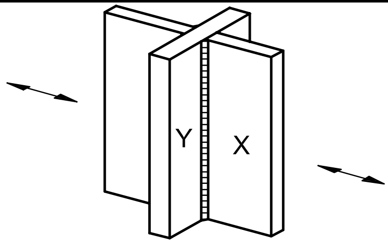
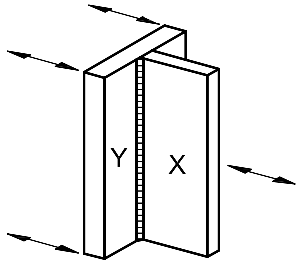
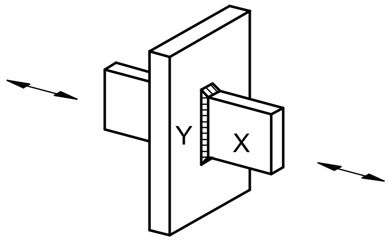
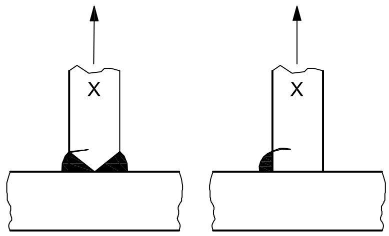
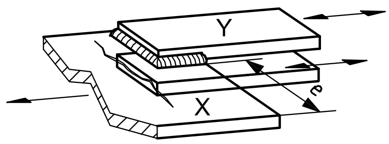
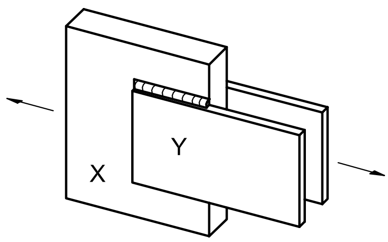
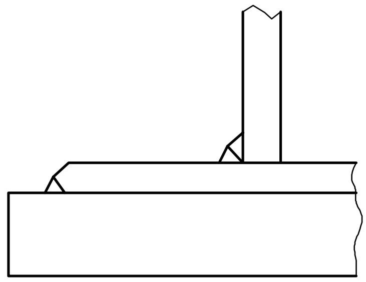
Type Number and description, explanatory comments	Class	Example, including failure mode
5.1 Parent metal adjacent to cruciform joints or T-joints (member marked X in sketches).		
a) Joint made with complete joint penetration groove welds and with any undercutting at the corners of the member dressed out by local grinding.	F	
b) Joint made with partial penetration or fillet welds with any undercutting at the corners of the member dressed out by local grinding. In this type of joint, failure is likely to occur in the weld throat unless the weld is made sufficiently large (to be checked with joint type 5.4).	F_2	
5.2 Parent metal adjacent to the toe of load-carrying fillet welds that are essentially transverse to the direction of applied stress (member X in sketch).		
a) Edge distance ⩾ 10 mm.	F_2	e = edge distance
b) Edge distance < 10 mm. The relevant stress in member X should be calculated on the assumption that its effective width is the same as the width of member Y. These classifications also apply to joints with longitudinal welds only.	G	
5.3 Parent metal at the ends of load-carrying fillet welds that are essentially parallel to the direction of applied stress, with the weld end on plate edge (member Y in sketch).	G	

Table A.16.10-11 (*continued*)

Type Number and description, explanatory comments		Class	Example, including failure mode
5.4	Weld metal in load-carrying joints made with fillet or partial joint penetration groove welds, with the welds either transverse or parallel to the direction of applied stress (based on nominal shear stress on the minimum weld throat area). This includes welds in doubler plates, anodes, pipe supports. In these cases, an assessment of the stress range in the weld throat should be made. In each case, the thickness correction should be based on the weld throat minimum thickness	W_1	

Remarks on potential modes of failure

Failure in cruciform or T-joints with complete joint penetration groove welds will normally initiate at the weld toe, but in joints made with load-carrying fillet or partial penetration groove welds, cracking may initiate either at the weld toe and propagate into the plate or at the weld root and propagate through the weld. In welds parallel to the direction of the applied stress, however, weld failure is uncommon; cracks normally initiate at the weld end and propagate into the plate perpendicular to the direction of applied stress. The stress concentration is increased and the fatigue resistance is therefore reduced, if the weld end is located on or adjacent to the edge of a stressed member rather than on its surface.

Table A.16.10-12 — Type 6: details in welded girders and tubulars

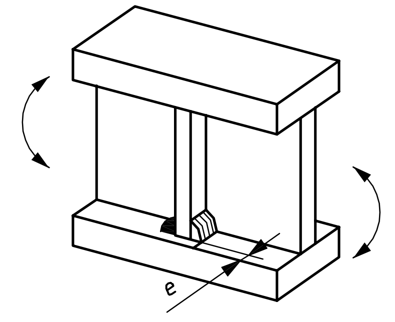
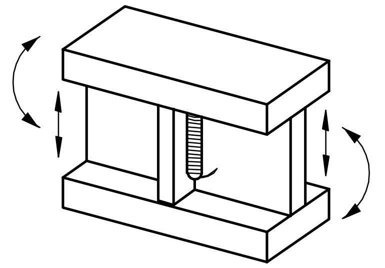
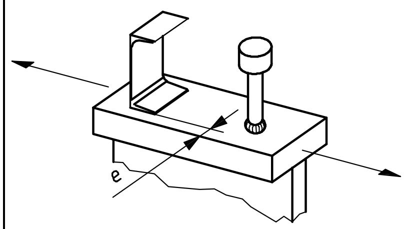
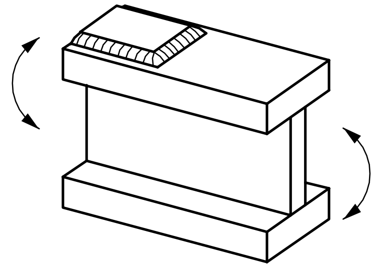
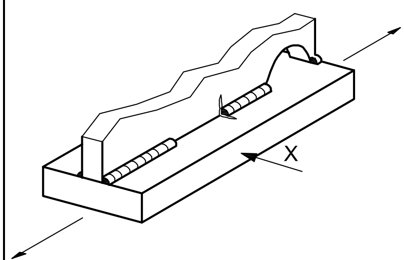
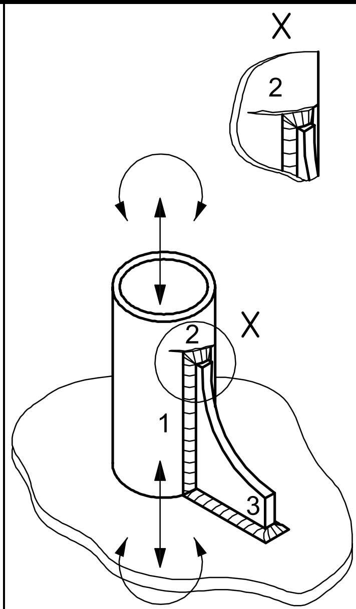
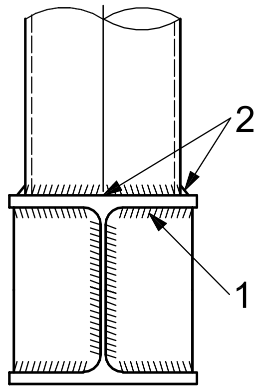
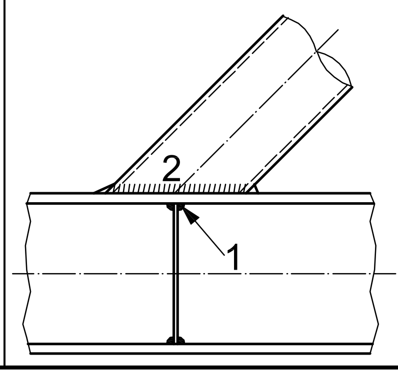
Type number and description, explanatory comments	Class	Example, including failure mode
6.1 Parent metal at the toe of a weld connecting a stiffener, diaphragm, etc, to a girder flange. a) Edge distance ⩾ 10 mm. b) Edge distance < 10 mm. Edge distance refers to distance from a free, i.e. unwelded, edge. In this example, therefore, it is not relevant, as far as the (welded) edge of the web plate is concerned.	 F G	 *e* = edge distance
6.2 Parent metal at the end of a weld connecting a stiffener, diaphragm, etc, to a girder web in a region of combined bending and shear. This classification includes all attachments to girder webs.	E	
6.3 Parent metal adjacent to welded shear connectors. a) Edge distance ⩾ 10 mm. This classification includes pile-to-sleeve shear connectors. b) Edge distance < 10 mm (see type 4.c).	 F G	 *e* = edge distance
6.4 Parent metal at the end of a partial length welded cover plate, regardless of whether the plate has square or tapered ends and whether or not there are welds across the ends. The class includes cover plates which are wider than the flange. However, such a detail is not recommended because it will almost inevitably result in undercutting of the flange edge where the transverse weld crosses it, as well as involving a longitudinal weld terminating on the flange edge and causing a high stress concentration.	G	
6.5 Parent metal adjacent to the ends of discontinuous welds, e.g. intermittent web/flange welds, tack welds unless subsequently buried in continuous runs. The relevant stress is the nominal stress in the member marked X in the sketch. If the weld is load-carrying, it should be checked with joint type 5.4. The same, adjacent to cope holes. The existence of the hole is allowed for in the joint classification; it should not be regarded as an additional stress concentration.	F	

Table A.16.10-12 (*continued*)

Type number and description, explanatory comments	Class	Example, including failure mode
6.6 Gusseted connections When the connection is subjected to significant forces, the stress range should be that in the tubular member, the gusset or the weld, including an SCF which arises from the overall form of the joint. To reduce the SCF, it is recommended in example 1 to slot the tubular and place the gusset through. a) Parent metal of gusset plate adjacent to a member or weld in full penetration welds connecting a gusset plate to a member — Location 1. c) Parent metal of a member adjacent to fillet, full or partial penetration welded gusseted connection — Location 2. Full penetration welds are normally required in such joints. d) Weld metal in fillet or partial penetration welds attaching a gusset plate to a member — Location 3.	 F F W_1	  

Remarks on potential modes of failure

Fatigue cracks generally initiate at weld toes and are especially associated with local stress concentrations at weld ends, short lengths of return welds, and changes of direction. Concentrations are enhanced when these features occur at or near an edge of a part (see remarks on Table A.16.10-10).

General comments

Most of the joints in this table are also shown, in a more general form, in Table A.16.10-10; they are included here for convenience as being the joints that occur most frequently in girder and tubular joints.

A.16.11 Fatigue resistance of the material

A.16.11.1 Basic S–N curves

The fatigue assessment of welded joints is based on the assumption that the connection has full-penetration single or double sided welding, unless otherwise stated.

Offshore structures are subjected to fatigue due to variable amplitude stresses. However, the prediction of fatigue damage under variable amplitude stresses is a complex subject and the most commonly adopted approach for the assessment of offshore structures is the use of the Palmgren-Miner summation law in combination with fatigue resistance in the form of an S–N curve for constant amplitude stresses. A limited number of variable amplitude fatigue tests on tubular joints have been undertaken and the results compared with constant amplitude S–N curves using an equivalent stress range, which has been defined as the cube root of the average value of $(\text{stress})^3$. This indicates that Palmgren-Miner's sum for the mean S–N curve falls essentially within the range of 0,5 to 2,0, with an average value of 1,8. This is comparable with the results from constant amplitude tests. A significantly larger number of test results are available for plate joints, which give an average Palmgren-Miner's sum of 1,1.

The S–N curves for tubular joints and castings are based on a comprehensive review of fatigue data for both tubular and plated joints. The background information is presented in References [A.16.11-1] and [A.16.11-2]. The S–N curves apply to crack growth associated with through-thickness cracking.

The tubular joint S–N curves have been derived from an analysis of data on tubular joints manufactured using welds conforming to a standard flat profile as given in AWS [A.16.10-2]. Therefore, the fatigue recommendations apply to joints, which conform to this AWS standard flat profile.

US investigations in this field have been carried out by EWI [A.16.11-2] on behalf of API. Both the HSE [A.16.11-1] and EWI [A.16.11-2] investigations concur on the general form of the S–N curves. The basic S–N curves relate to in-air conditions. Separate curves are presented for joints in sea water with adequate corrosion protection (i.e. from −850 mV to −1 100 mV). Fatigue data for tubular joints indicate that, in general, there is a reduction in the fatigue performance in sea water under cathodic protection for stress range magnitudes corresponding with resistances less than 10^6 cycles for in-air conditions. For smaller stress range magnitudes and corresponding resistances greater than 10^6 cycles in-air the fatigue lives in sea water are restored to that of in-air conditions. References [A.16.11-3] and [A.16.11-4] present the results from fracture mechanics evaluations and illustrate the detrimental effect of sea water relative to air for joints with and without adequate cathodic protection. In normal design of offshore structures the most significant damaging stress range cycles correspond with resistances greater than $N = 10^6$ cycles, such that the detrimental reduction in fatigue life for cathodically protected joints is not apparent. The basic S–N curves given in Table 16.11-1 are therefore applicable for these conditions. In instances where significant damage occurs for stress range cycles corresponding with resistances less than $N = 10^6$ cycles, a penalty factor of 2 on N is recommended. For joints in freely corroding conditions, or for joints with corrosion protection levels more negative than −1 100 mV, a factor of 3 on N for all resistances, without a change of slope at 1×10^7 cycles, is recommended.

A number of tubular joints used in deriving the basic TJ S–N curve had chord and braces with equal diameters (i.e. $\beta = 1{,}0$). Some of these joints showed extensive weld inter-run cracking in preference to weld toe cracking. This can be significant in relation to the application of weld toe grinding improvement techniques, since clearly toe grinding of the chord or brace weld toes alone does not improve the fatigue performance of these joints. Improvement would only be achieved if the weld face is also toe ground to remove all of the inter-run toes. However, an assessment of the $\beta = 1{,}0$ joints, using the recommended SCF equations in A.16.10.2.2, indicated that the predicted fatigue lives are significantly above the TJ curve.

A.16.11.2 High strength steels

A limited amount of test data for plate joints with representative yield strengths up to 540 MPa [A.16.11-1] and tubular joints manufactured from some modern high strength steels [A.16.11-3] have suggested that the fatigue performance in sea water under cathodic protection and under free corrosion is similar to that for medium strength structural steels, thus allowing the S–N curves in Table 16.11-1 also to be used for these steels. Alternatively, test data or fracture mechanics analysis should be used to determine appropriate S–N curves.

High strength steels are increasingly being used in the fabrication of offshore structures, particularly for jack-up legs, which are made from steels with typical yield strengths of 700 MPa to 800 MPa. The effect of sea water on the fatigue performance of these materials is thought to be more detrimental than for medium strength structural steels because of their greater susceptibility to cracking from hydrogen embrittlement under variable stresses in sea water [A.16.11-3]. The susceptibility to hydrogen embrittlement increases with increasing yield strength and increasingly negative cathodic protection potential. A number of studies have identified excessively negative cathodic protection potential as a cause of cracking due to the generation of hydrogen which enhances crack growth rates at the crack tip [A.16.11-3]. Evidence of hydrogen cracking found in jack-ups during routine surveys has been reported in Reference [A.16.11-5]. It is therefore important that the fatigue performance of selected high strength steels is understood and that appropriate levels of cathodic protection are applied.

A.16.11.3 Cast joints

The S–N curve for cast joints has been derived from tests in air on large-scale cast joints with thicknesses in the range of 18 mm to 40 mm, tested principally at $R = -1$ (i.e. stresses varying from a minimum in compression to an equal maximum in tension), and cruciform specimens with thicknesses in the range of 38 mm to 125 mm tested at $R = 0$. Similar mean curves are obtained from the two sets of data using an inverse slope of 4 [A.16.11-1]. Since cast joints are stress relieved, the R -ratio has an influence on the fatigue behaviour. The S–N curve for the test data could therefore overestimate the fatigue performance of cast joints tested at $R > -1$. Hence, allowance has been made for the influence of mean stresses by applying a 20 % reduction to the maximum experimental stress range used to determine the CJ curve.

There is insufficient experimental evidence to justify a change in slope in the low stress/high cycle region, the highest experimental resistance being 5×10^6 cycles. A conservative approach of using a constant slope of $m = 4$ for all numbers of cycles is therefore recommended.

Fracture mechanics analysis shows that casting defects can have a significant effect on the fatigue resistance, and the design curve corresponds to four standard deviations in N below the mean curve to allow for the possibility of undetected defects [A.16.11-6]. The curve is applicable to castings, which satisfy defect acceptance criteria compatible with current offshore practice; Reference [A.16.11-1] contains further information.

In order to determine whether weld repairs could be detrimental to the fatigue performance of cast joints, fatigue tests on cruciform specimens in both air and sea water were undertaken [A.16.11-7]. These tests showed that, provided weld repaired surfaces are ground flush to the as-cast profile and free from weld toe defects, the CJ S–N curve may be used for cast joints with weld repairs; see also Reference [A.16.11-1].

The limited data for castings in sea water (under conditions of cathodic protection and free corrosion) suggest that the environmental reduction factors on resistance from A.16.11.1 should be used.

The fatigue assessment of cast joints requires that a finite element analysis is performed to determine the location of the maximum stress range in the casting. For cast tubular nodal connections, the brace to casting circumferential butt weld can be the most critical location.

A.16.11.4 Thickness effect

An assessment by HSE [A.16.11-1] and EWI [A.16.11-2] of a wide range of data for various combinations of member forces and/or moments has shown that the fatigue performance is dependent on the member thickness, the performance decreasing with increasing thickness for the same stress range.

The TJ and OJ representative curves are based on a material thickness of 16 mm. EWI derived a thickness exponent of 0,29, compared with 0,3 in the HSE investigations. Given the similarity of these values, an exponent of 0,3 is specified.

The basic material thickness for the CJ representative curve is 38 mm. Fracture mechanics predictions [A.16.10-2] show that the thickness effect in castings is smaller than in welded joints and an exponent of 0,15 is specified.

A.16.12 Fatigue assessment

A.16.12.1 Cumulative damage and fatigue life

The Palmgren-Miner rule is not precise. Its accuracy depends on issues such as stress range history, type of constructional detail and failure definition. For tubular joints in offshore structures, the Palmgren-Miner rule can be somewhat unconservative. However, this unconservatism is generally more than compensated by bias (and scatter) associated with the geometric stress range and the S–N curve. Hence, adjustment of the Palmgren-Miner rule is not considered necessary at this time.

Fatigue life estimates should be viewed with scepticism. The S–N curves used in design are representative curves (e.g. 95 % confidence of 97,5 % survival), as is typical of land based structures like bridges. This survival level implies that there is little chance of failure at the predicted life. An estimate of mean fatigue life using the mean S–N curve is generally several times the calculated life using the design S–N curve. In addition to uncertainties associated with wave action, the primary uncertainties and probable conservatisms in fatigue life estimates stem from

a) the geometric stress range history, and

b) the S–N curve assumption in the low-S–high-N (the very long-life) regime, for which there are few supporting data.

The reliability of fatigue damage estimates is not yet established because full calibration of the analysis techniques, including both failures and non-failures, has not broadly occurred.

Fatigue life estimates of very short duration are even more doubtful when they are based on long-term wave histories, taken over a period of years. Short-term conditions can easily deviate from average conditions and can be better or worse than average.

Fatigue life estimates can be evaluated in a probabilistic framework. For more information, the user should refer to References [A.16.12-1] and [A.16.12-2]. However, broad ranges of probabilities are possible. The uncertainties mentioned above affect probabilistic as well as deterministic fatigue life estimates. Probability estimates are most useful when used in relative terms.

For structures that need their life to be extended or are being re-used or converted to a new application, prior damage [D_1 in Equation (16.12-3)] should be estimated via inspection findings. An absence of crack discoveries should *not* be assumed to mean no prior damage has occurred. For example, if magnetic particle inspection (MPI) reveals no evidence of defects, it may be assumed that the prior damage in terms of the Palmgren-Miner sum is limited to 0,3 for a welded tubular joint and to 0,5 for a welded plate detail. Assuming a defect-free inspection, lower values of assumed damage can be justified by the designer based on analysis, if the prior history of the structure can be established with confidence. However, a value of zero is normally only used for those details that will be modified so as to eliminate prior damage.

A.16.12.2 Fatigue damage design factors

Fatigue damage design factors for fatigue of steel components primarily depend on failure consequence and in-service inspectability. Failure criticality is normally established on the basis of redundancy analyses. A structure with redundancy, capability for in-service inspection and the possibility for repair/strengthening is preferred, especially for the design of a new structural concept or a conventional structure for new environmental conditions. Joints in the splash zone are considered as “not inspectable”.

In lieu of more detailed assessment, the fatigue damage design factors can be taken from Table A.16.12-1.

Table A.16.12-1 — Fatigue damage design factors, γ_{FD}

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

Both the factor of 5 and the factor of 10 imply that a significant change in fatigue reliability only occurs when there is a significant change in the predicted life (or Palmgren-Miner damage sum) for the design service life of the structure. As the slopes of the S–N curves vary between 3 and 5, values $1 < \gamma_{FD} < 10$ are equivalent to much smaller changes in the calculated stresses of from 1,38 S to 1,71 S for a factor of 5 and from 1,58 S to 2,15 S for a factor of 10.

The factors given in Table A.16.12-1 should be considered to relate to exposure level L1, but should also be used for exposure levels L2 and L3. There is currently insufficient background to establish different factors for lower exposure levels.

The fatigue damage design factors do not differentiate between fatigue analysis procedures. At present, there is little certainty in how the various procedures compare in terms of reliability, so the same set of explicit fatigue damage design factors is generally applied to all of them. The fatigue damage design factors also do not differentiate such aspects as risk to assets and difficulties associated with repairs, or lost production due to repairs. It is the owner's responsibility to assess how these sorts of risk should be addressed in the design phase.

There are instances where the cited fatigue damage design factors may be reduced. An example could be a structural component above water, for which inspection can be either easier or more frequent. A reduction in fatigue damage design factor can also be appropriate if loss of the component does not jeopardize personnel safety or the environment. Finally, smaller fatigue damage design factors can be justified if the fatigue analysis algorithm has been calibrated to the structural type and design situations being considered. However, use of reduced factors should generally require some form of documented justification.

In selecting fatigue damage design factors, inspectability and inspection technique need careful consideration. In general, the in-service inspection for fatigue damage should be more thorough than the general level II survey described in Clause 23. Locations below 200 m under the water surface should generally be assumed to be uninspectable. For some complex joints, such as internally stiffened ones, cracking can originate from the inside (hidden) surfaces. Hence, the need and possibility for inspection prior to crack penetration through the thickness should be considered at the design stage. A trade-off exists between e.g. introducing a fatigue damage design factor of 5 (for a component that is not failure critical) and inspecting in-service with a more complex technique such as ultrasonics. As ultrasonic inspection normally requires the use of diving technicians, the risk of diving should be included in the considerations.

Even though a given component is considered readily inspectable from exposed surfaces, inspection frequency should still be balanced with the fatigue damage design factor. In some structural components, such as those with low SCFs, the rate of crack growth can surpass normal circumstances and inspection intervals.

Despite the need to address inspectability during the design phase, there is no implied requirement to perform a regular, detailed inspection of each and every joint for which a fatigue damage design factor from the inspectable category is adopted. The scope and frequency associated with the inspection plan involve considerations that extend well beyond the issue of the fatigue analysis recipe alone. However, if no inspection is clearly intended from the start for a particular class of joint, then the fatigue damage design factor should be selected from the not inspectable category.

A.16.12.3 Local experience factor

The default value of the local experience factor, k_{LE}, is1,0. (See the regional information in Annex H for some cases where available data supports the use of a factor that is different from 1,0.) In other cases, a factor different from 1,0 should be substantiated by reliable evidence and should be fully documented.

A local experience factor may be used for the fatigue life estimation of a new structure design, for the extension of the fatigue life of an existing structure, and for an assessment of the fatigue life of a structure that is re-used or converted to a new application. If such a structure has accumulated prior fatigue damage, any local experience may only be used for an assessment of the remaining fatigue life.

As the fatigue process is complex and fatigue damage is sensitive to many different factors, local experience with existing structures can only be considered relevant for the fatigue assessment of a structure under consideration if the following conditions are satisfied:

- there is a database with fatigue data from existing structures in-place that have not suffered from noticeable fatigue damage during time periods exceeding the predicted fatigue lives for these structures;
- the environment to which the in-place structures are subjected is essentially the same as the environment that the structure under consideration will be subjected to; this applies in particular, but not exclusively, to the wave environment;
- the structure under consideration is of the same general type, is of similar configuration and has comparable dynamic characteristics as existing structures;
- the structure under consideration contains similar constructional details as existing structures;
- the structure under consideration is of similar material and fabricated in a comparable manner as existing structures.

A.16.13 Other causes of fatigue damage than wave action

A.16.13.1 General

No guidance is offered.

A.16.13.2 Vortex induced vibrations

No guidance is offered.

A.16.13.3 Wind induced vibrations

Where necessary, measures should be taken to avoid wind induced vibrations of members during fabrication. If this is not possible, or the measures are not successful, the resulting fatigue damage during the fabrication period for the members involved should be estimated and included in the overall fatigue damage assessment.

A.16.13.4 Transportation

Guidance on fatigue assessment during transportation is given in Reference [A.16.13-1]. The damage assessment during transportation should use reliable statistical information on the wave conditions along the tow route in the expected tow season. Normally, these should be provided in the form of wave scatter diagrams.

Spectral analysis techniques should be used to account for the random nature of the wave environment and for compatibility with standard vessel motion prediction techniques. The motions of the structure during the tow should be evaluated using either model tests or a suitable vessel motions analysis program. The possibility of different wave directions relative to the tow occurring should be taken into account. Structural analyses should be performed to evaluate the stresses due to motions and deformations during tow conditions. For structures transported on barges, the stiffnesses of the barge, structure and sea-fastening should be taken into account.

As the actual weather conditions experienced during the tow can deviate significantly from the statistical information for the tow season, no matter how reliable this information is, it is recommended that sensitivity analyses be performed using a modified scatter diagram or diagrams. The modified scatter diagram(s) could, for example, include one occurrence of the most severe sea state that can occur along the tow route during the tow period, increased probabilities of occurrence of lesser sea states, and/or increased probabilities of occurrence of unfavourable wave directions relative to the tow. If the sensitivity analyses indicate that fatigue damages during transportation are always acceptable, no further consideration is necessary. However, if the sensitivity analyses suggest that unacceptable fatigue damage can occur further consideration of the transportation phase is warranted.

A.16.13.5 Installation

Driving fatigue should be considered for piles and driven caissons/monotowers.

Design measures to minimize potential fatigue damage during pile driving include

- relocating attachments to avoid the sensitive areas,
- using substantial connections for those attachments which cannot be relocated,
- using doubler plates for connections to main members, and
- using welded rather than screwed or bolted fittings for piping.

A.16.13.6 Risers

No guidance is offered.

A.16.14 Further design considerations

A.16.14.1 General

No guidance is offered.

A.16.14.2 Conductors, caissons and risers

No guidance is offered.

A.16.14.3 Miscellaneous non-load carrying attachments

No guidance is offered.

A.16.14.4 Miscellaneous load carrying attachments

Where attachments are located close to supporting structural members, the effect of flow enhancement of the free stream flow should be taken into account.

Where doubler plates are used, these should be designed such that the connection of the attachment to the doubler plate fails before the connection of the doubler plate to the primary member.

A.16.14.5 Conical transitions

Guidance on SCFs for unstiffened conical transitions is given in 13.6.3.5. Guidance on SCFs for stiffened conical transitions is given in Reference [A.16.14-1].

A.16.14.6 Members in the splash zone

Plan framing levels immediately below or above the water surface are immersed and emerged during the passage of waves exceeding a certain threshold height. Normal wave action on these members is highly non-linear and is most likely not reliably represented in the spectral or other analysis methods. The manner in which the action on these members is included will differ from program to program and should be carefully reviewed. A separate evaluation based on deterministic analysis principles using estimated actions and numbers of cycles for those waves that exceed the threshold should be performed to account for these effects.

Variations in buoyancy due to the passage of waves can also cause cyclic forces in members in the splash zone. If these buoyancy variations are not explicitly represented in the calculation algorithms of the computer program used, the effect of variable buoyancy actions on member stresses should be assessed and included, if significant, before fatigue damage calculations are performed.

Members located near the water surface can further be subjected to wave slamming actions in addition to the normal wave actions that are included in the fatigue analysis. The slamming actions cause vibrations of the member and are largest for horizontal members lying parallel to the wave crest. Guidance on estimating slamming actions can be found in the general literature. Where relevant, the fatigue damage associated with vibrations due to slamming should be included in the assessment.

A.16.14.7 Topsides structure

No guidance is offered.

A.16.14.8 Inspection strategy

During the design of new structures, it is common to make initially conservative assumptions (e.g. with respect to SCF or S–N curve selection) when performing the fatigue analysis to screen the structure with a view to identifying potential problem areas. These areas are then examined in more detail by refining the analysis assumptions and eliminating conservatism where possible and justified. Consequently, the ranking of calculated fatigue lives does not necessarily reflect the true ranking if all structural details are not refined to a common level. This observation should be considered if fatigue lives are used for prioritizing areas of the structure for inspection.

Similarly, the approximations in the analysis procedure should be reviewed if low calculated fatigue lives based on approximate methods of analysis are used as the basis for an extensive inspection effort. In such cases, consideration should be given to performing a spectral analysis to refine the calculated lives where possible in order to try and reduce the inspection effort.

Generally, an initial inspection interval of one quarter of the calculated fatigue life is recommended for flat plate and welded tubular joints. Consequently, joints with calculated fatigue lives of up to four times the structure design service life would normally require inspection for fatigue once during the design service life.

A.16.15 Fracture mechanics methods

A.16.15.1 General

The benefits of defect assessment procedures (see, for example, Reference [A.16.15-1]) for the fitness-for-purpose assessment of offshore structures are widely recognized, and defect assessment is increasingly being used in design and during fabrication and in-service inspection. However, existing procedures are based on general principles, and their application to tubular joints — which is a particularly complex problem due to the nature of the actions and the geometry of the joint — requires special care. Detailed information is given in References [A.16.15-2], [A.16.15-3] and [A.16.15-4].

A.16.15.2 Fracture assessment

Static failure can occur by fracture or by plastic collapse, if a crack propagates to a critical size. Failure assessment procedures combining considerations of failure by fracture and by plastic collapse are given in Reference [A.16.15-1]. The fracture assessment should be performed for each increment of crack growth.

The possibility of a fracture failure is normally greater in the brace than in the chord, since there is a greater likelihood of the crack propagating into material with lower toughness in the weld region. Further details can be found in Reference [A.16.15-1].

A.16.15.3 Fatigue crack growth law

Recommended values for the fatigue crack growth rate parameters, C and m, are presented in Reference [A.16.15-5].

The complexity of the normalized stress intensity factor (Y) usually requires the use of numerical integration methods. It is important that a sensitivity analysis is performed to establish that convergence is achieved using the selected increment of crack growth.

Knowledge of the initial crack (defect) size is essential in any fracture mechanics analysis. The predicted number of cycles to failure is sensitive to this parameter. It can be necessary to perform a sensitivity analysis to determine the significance of the assumed initial defect size.

For assessments performed during design, it is normal to assume that the initial crack (defect) is located at the position of the maximum GSR. The assumed initial crack (defect) size, a_{i}, should be based on the accuracy of the non destructive testing (NDT) method used during fabrication. Inspection data can be highly variable and there is an increasing tendency towards the use of probability of detection (POD) curves. Where there is sufficient information, the minimum detectable defect size may be based on the 90 % POD, 95 % confidence level. However, this depends on the inspection method used and the selection of the minimum detectable defect size requires specialist advice.

The final crack size, a_{f}, is limited to the critical crack size in view of either fracture toughness or plastic collapse; see A.16.15.2.

The assessment of a crack that is detected during inspection requires information on the crack location, depth from the surface and length along the surface of the material.

A.16.15.4 Stress intensity factors

The principal sources of normalized stress intensity factors, Y, are

- standard solutions for semi-elliptical cracks in plates (see, for example, Reference [A.16.15-5]) used in conjunction with magnification factors and the moment release method,
- numerical methods by FEA or boundary element analysis,
- analytical methods (e.g. using weight functions), and
- empirical methods, involving the assessment of fatigue crack growth rate data.

The use of plate solutions is generally conservative, particularly for deep cracks, but provides stress intensity factors for a very wide range of parameters. The use of the moment release method in conjunction with plate solutions reduces the excessive conservatism and this approach is recommended for general use. The moment release method assumes that the net force acting across the cross-section of a cracked tubular member in an offshore structure, and hence the stress intensity factor, is reduced by the redistribution of forces. The linear moment release method involves the reduction of the bending stress component using the following simple expression:

$$\sigma_{\mathrm{b,c}} = \sigma_{\mathrm{b}}\left(1 - a/T\right) \tag{A.16.15-1}$$

where $\sigma_{\mathrm{b,c}}$ and σ_{b} are the bending stress components in the cracked and uncracked joint, respectively, and T is the wall thickness.

The availability of stress intensity factor solutions for welded joints is limited. In Reference [A.16.15-1], the stress distribution through the wall thickness is approximated to a combined bending and membrane stress field; further known stress intensity factor solutions for cracks in plates (see, for example, Reference [A.16.15-5]) are used in conjunction with a magnification factor to account for the influence of the stress concentration at the weld toe.

Magnification factors have been derived from two-dimensional FEA of butt and fillet welded joints and this approach is generally conservative. Alternatively, stress intensity factors can be evaluated by analysis or by approximate analytical methods, e.g. weight function analysis.

Models which take account of the variation of the GSR around the brace/chord weld periphery have also been developed [A.16.15-6], but validation is limited.

Below a certain threshold value of the stress intensity factor range (ΔK), no crack growth will occur. Recommended threshold values (ΔK_{th}) for carbon and carbon manganese steels in air as well as in sea water with cathodic protection are $\Delta K_{th} = 170$ Nmm$^{-3/2}$ at $R = 0$ (tensile stress ranges between zero and a maximum stress) and $\Delta K_{th} = 63$ Nmm$^{-3/2}$ at $R = 0{,}5$ (tensile stress ranges between half the maximum and the full maximum stress). For sea water with free corrosion, a threshold stress intensity factor range $\Delta K_{th} = 0$ is recommended.

A.16.15.5 Fatigue stress ranges

For a fatigue crack growth analysis, the full stress range should always be used, even when the calculated stress range cycle is partially compressive, since the stress range can become effectively tensile due to the effects of mean and residual stresses.

A.16.15.6 Castings

The through-thickness stress distribution should be determined using FEA. The fatigue crack growth rate data for typical structural steels are generally found to be also applicable to cast material. However, where significant differences in chemical composition or mechanical properties exist, it can be necessary to obtain specific fatigue crack growth rate data.

A.16.16 Fatigue performance improvement of existing components

A.16.16.1 General

Post-weld improvement techniques may be used to improve fatigue performance. These techniques, discussed below, improve fatigue performance by improving the local geometry at the weld toe, by reducing the stress concentrations and/or by modifying the residual stresses. All these effects are included in the improvement factors on fatigue performance for each technique. Specific requirements for the various techniques are noted or referenced below.

The GSR to be used for an assessment of the improved performance should be obtained from equivalent joints before the improvement technique is applied, from FEA or from SCF equations. They should not be obtained from measurements on improved joints.

The designer should be wary when applying weld improvement techniques, especially a powerful one like peening. If later fatigue cracking occurs, it should *not* be expected to originate at the treated location. However, if cracking does originate at a treated weld toe, the life associated with subsequent propagation is likely to be much shorter than is normal for untreated details.

A.16.16.2 Post-weld heat treatment

As-welded joints contain significant tensile residual stresses induced by the welding process, which combine with the operating stresses to promote fatigue failure. This is due to the increase of the effective mean stress and, for situations where the stress range consists of a compressive component, of the effective stress range. It follows that the reduction of tensile residual stresses can increase the fatigue performance.

A comparison of the fatigue behaviour of as-welded and post-weld heat treated joints has confirmed that post-weld heat treatment (PWHT) can have a beneficial effect on the fatigue behaviour of welded joints. However, the effect of PWHT diminishes with increasing R - ratio and is negligible at $R > 0$ (all tensile stress ranges). Thus, the fatigue performance of post-weld heat treated and as-welded joints at R-ratios greater than zero are very similar and the same S–N curves apply.

A significant drawback of accepting PWHT as a fatigue improvement measure in design is that knowledge of the residual stress distribution, including the contribution of long-range fit-up stresses, is required. This information is not usually available.

A.16.16.3 Weld profiling

Limited investigations of the influence of weld profile on the fatigue resistance of tubular joints have shown there is no clear evidence that weld profiling leads to improved fatigue performance [A.16.16-1]. The basic TJ S–N curve has been derived from an analysis of data on tubular joints manufactured using welds conforming to a standard flat profile given in AWS [A.16.10-2]. Therefore, the fatigue recommendations apply to joints, which conform to this AWS standard flat profile.

Improvements through any form of profiling can be justified using information from an appropriate test programme for tubular joints for the condition being considered.

A.16.16.4 Weld toe grinding of tubular joint welds

For welded joints in-air and for joints in sea water with adequate cathodic protection, the fatigue performance can be increased by controlled local machining or grinding to produce a smooth concave profile at the weld toe. This is especially beneficial at low stress ranges. Experimental data indicate that this technique can lead to an increase in the fatigue performance by a factor of approximately 2. It should be noted that the beneficial effect of weld toe grinding can be reduced by free corrosion, though it tends to be restored by cathodic protection, see References [A.16.16-2] and [A.16.16-3].

A limited number of tests have demonstrated the importance of quality control of the grinding procedure. The grinding procedure should ensure that all defects in the weld toe region have been removed by grinding to a depth not less than 0,5 mm below the bottom of any visible undercut or defect. The maximum depth of local grinding should not exceed 2 mm or 5 % of the plate thickness, whichever is less. NDT of the joint is required after grinding to verify that no significant defects remain and, for fillet welded connections, it is important that the required throat size is maintained. Further quality control aspects apply, particularly on the shape of grinding burrs and their orientation; see Reference [A.16.16-2].

A.16.16.5 Grinding of butt welds

For butt welded connections, additional benefit can be gained by grinding the weld cap flush. The effect of this is to improve the joint classification (see Table A.16.10-9).

A.16.16.6 Hammer peening

By hammer peening the toes of welded connections, surface defects can be eliminated, the transition between the parent and weld material is smoothed out, and beneficial compressive residual stresses are induced at the surface, all of which contribute to the enhancement of the fatigue performance of the treated weld. The net effect is to delay crack development and retard or eliminate growth of cracks already present.

The objective in hammer peening is to obtain a smooth groove at the weld toe. The grooved depth should be at least 0,3 mm, but should not exceed 0,5 mm [A.16.16-4], [A 16.16-5]. The equipment and procedure required to attain this groove configuration should be established via trials on detail mock-ups. Note that the number of passes required is determined by the equipment and procedure. Heavy duty pneumatic hammers are preferred. The bit tip radius should be about 3 mm, so as to expedite the process and facilitate treatment right at the weld toe. Extensive use of peening has ergonomic implications. Consideration should be given to limiting the consecutive hours spent peening by one individual and vibration dampening gloves should be used. Peening can result in metal rollovers along the sides of the groove. Tests have shown that such rollovers can be deleterious to fatigue performance and should be removed[16.16-6]. Removal eliminates difficulty with interpretation of later inspection findings. Peened weld toes should be inspected by MPI directly after peening and any burr grinding.

The recommended fatigue performance improvement factor is 4. This value is significantly less than that found in many test programmes, and varies with stress range magnitude and other variables. The reduced value takes into account uncertainties in

a) mean stress,

b) dominant stress range magnitude, and

c) the effects of overloads.

The fatigue performance improvement factor may be applied to both tubular and non-tubular weld details. At nodes constructed by nodal fabrication, the effectiveness of peening is improved if it is performed after fit-up of the associated brace.

The benefits of hammer peening on fatigue performance can only be realized through adoption of adequate quality control procedures. References [A.16.16-4] and [A.16.16-5] contain the state-of-the-art in this field, and should be consulted in the preparation of adequate QC procedures prior to taking benefit for fatigue performance enhancement.

A.17 Foundation design

A.17.1 General

A.17.1.1 Applicability

No guidance is offered.

A.17.1.2 Overall considerations

No guidance is offered.

A.17.1.3 Exposure levels

The reasons for the distinction between new and existing structures include the following.

a) The reliability of the foundation design depends on the type of soil, its variability, the amount of data collected and the suitability of the design method to the specific conditions. Experience indicates that there is a tendency for less data to be available at the design stage for exposure level L2 and exposure level L3 structures than for exposure level L1 structures. For existing structures, installation records, e.g. pile driving records, generally provide further relevant data for exposure level L2 and L3 structures.

b) If a foundation component such as a pile fails it will probably cause the structure to become unserviceable. Repair is likely to be impossible, and at best will be difficult and expensive. Thus the balance between reduced cost and higher reliability normally favours reliability. In particular, exposure level L2 and L3 structures frequently have non-redundant foundations and the failure of one pile can lead to collapse. However, for existing structures the cost and uncertainty of intervening in the foundation systems puts the balance in favour of reduced reliability for exposure level L2 and L3 structures.

A.17.2 Pile foundations

A.17.2.1 Types of pile foundation

No guidance is offered.

A.17.2.2 Driven piles

No guidance is offered.

A.17.2.3 Drilled and grouted piles

There are two types of drilled and grouted piles.

a) Single-stage piles

For the single-stage drilled and grouted pile an oversized hole is drilled to the required penetration, a pile is lowered into the hole and the annulus between the pile and the soil is grouted. This type of pile can be installed only in soils which will hold an open hole to the surface. As an alternative method, the pile with expendable cutting tools attached to the tip can be used as part of the drill string to avoid the time required to remove the drill bit and insert a pile.

b) Two-stage piles

The two-stage drilled and grouted pile consists of two concentrically placed piles grouted to become a composite section. A pile is driven to a penetration which has been determined to be achievable with the available equipment and below which an open hole can be maintained. This outer pile becomes the casing for the next operation, which is to drill through it to the required penetration for the inner or “insert” pile. The insert pile is then lowered into the drilled hole, and the annuli between the insert pile and the soil and between the two piles are grouted. The diameter of the drilled hole should be at least 150 mm (6 in) larger than the insert pile diameter.

A.17.2.4 Belled piles

No guidance is offered.

A.17.2.5 Vibro-driven piles

No guidance is offered.

A.17.3 General requirements for pile design

No guidance is offered.

A.17.4 Pile capacity for axial compression

A.17.4.1 General

No guidance is offered.

A.17.4.2 Representative axial pile capacity

It is not always correct to add the representative value of the end bearing to the representative value of the skin friction to obtain the representative value of the axial capacity of a pile. This subject is addressed in References [A.17.4-1], [A.17.4-2] and [A.17.4-3]. For the particular case of a belled pile, this matter is discussed in Reference [A.17.4-3].

A.17.4.3 Skin friction and end bearing in cohesive soils

A.17.4.3.1 General

Estimating pile capacity in clay soils requires considerable judgment in selecting design parameters and in interpreting calculated capacities. Some of the items that should receive design consideration are detailed in A.17.4.3.2 and A.17.4.3.3.

A.17.4.3.2 Axial pile capacity in clay

a) Load test database for piles in clay

A number of studies[A.17.4-4] to [A.17.4-9] have been carried out aimed at collecting and comparing axial capacities from relevant pile load tests to those predicted by traditional offshore pile design procedures. Studies such as these can be very useful in tempering one’s judgment in the design process. It is clear, for example, that there is considerable scatter in the various plots of measured vs. predicted capacities. The designer should be aware of the many limitations of such comparisons when making use of these results. Limitations of particular importance include the following.

1) There is considerable uncertainty in the determination of both predicted capacities and measured capacities. For example, determination of the predicted capacities is very sensitive to the selection of the undrained shear strength profile, which itself is subject to considerable uncertainty. The measured capacities are also subject to interpretation as well as possible measurement errors.

2) The conditions under which the pile load tests are conducted generally vary significantly from the design actions and field conditions. One clear limitation is the limited number of tests on deeply embedded, large diameter, high capacity piles. Generally, pile load tests have capacities that are 10 % or less of the prototype capacities. Another factor is that the rate of change and the cyclic history of the actions are usually not well represented in the load tests (see A.17.6.3). For practical reasons, the pile load tests are often conducted before full set-up occurs (see A.17.4.3.3). Furthermore, the pile tip conditions (closed vs. open-ended) can differ from offshore piles.

3) In most of the studies an attempt has been made to eliminate those tests that are thought to be significantly affected by extraneous conditions in the load test, such as protrusions on the exterior of the pile shaft (weld beads, cover plates, etc.), installation effects (jetting, drilled out plugs, etc.), and artesian conditions, but it is not possible to be absolutely certain in all cases.

The database includes a number of tests that were specially designed for offshore applications as well as a number of published tests that are fortuitously relevant to offshore conditions (appropriate pile type, installation method, soil conditions, etc.). The former are generally higher quality and larger scale, and hence are particularly important in calibrating the design method. The tests most relevant to offshore applications have all been conducted in the United States or in Europe. As regional geology and particularly operating experience are considered very important in foundation design, care should be exercised in applying these results to other regions of the world. In addition, the designer should note that certain important tests in silty clays of low plasticity, such as at the Pentre site[A.17.4-10] indicate overprediction of frictional resistance by the Equations (17.4-2) to (17.4-4). The reason for this overprediction is not well understood and has been an area of active research. The designer is thus cautioned that pile design for soils of this type should be given special consideration.

Additional considerations that apply to drilled and grouted piles are discussed in References [A.17.4-11] and [A.17.4-12].

b) Alternative methods of determining pile capacity

Alternative methods of determining pile capacity in clays exist, based on sound engineering principles and consistent with industry experience, and may be used in practice. One such method is described below.

For piles driven through clay, f may be less than or equal to, but should not exceed, the undrained shear strength of the clay, c_u, as determined by unconsolidated-undrained (UU) triaxial tests and miniature vane shear tests.

Unless test data indicate otherwise, f should not exceed c_u or the following limits.

1) For highly plastic clays, f may be equal to c_u for underconsolidated and normally consolidated clays. For overconsolidated clays, f should not exceed 48 kPa (1 kips/ft^2) for shallow penetrations or the equivalent value of c_u for a normally consolidated clay for deeper penetrations, whichever is greater.

2) For other types of clay:

$$f = c_u \quad \text{for } c_u < 24 \text{ kPa } (0{,}5 \text{ kips/ft}^2) \tag{A.17.4-1a}$$

$$f = c_u/2 \quad \text{for } c_u > 72 \text{ kPa } (1{,}5 \text{ kips/ft}^2) \tag{A.17.4-1b}$$

f varies linearly for values of c_u between the above limits.

For other methods, see References [A.17.4-4], [A.17.4-5], [A.17.4-6] and [A.17.4-8].

It has been shown[A.17.4-9] that, on the average, the above cited methods predict the available but limited pile load test database results with comparable accuracy. However, capacities for specific situations computed by different methods can differ by a significant amount. In such cases, pile capacity determination should be based on engineering judgment, which takes into account site-specific soils information, available pile load test data, and industry experience in similar soils.

c) Establishing design strength and effective overburden stress profiles

The axial pile capacity in clay determined by these procedures is directly influenced by the undrained shear strength and effective overburden stress profiles selected for use in analyses. The wide variety of sampling techniques and the potentially large scatter in the strength data from the various types of laboratory tests complicate appropriate selection.

UU triaxial compression tests on high quality samples, preferably taken by pushing a thin-walled sampler with a diameter of 75 mm (3 in) or more into the soil, are recommended for establishing strength profile variations because of their consistency and repeatability. In selecting the specific shear strength values for design, however, consideration should be given to the sampling and testing techniques used to correlate the procedure to any available relevant pile load test data. The experience with pile performance is another consideration that can play an important role in assessing the appropriate shear strength interpretation.

Miniature vane tests on the pushed samples should correlate well with the UU test results and will be particularly beneficial in weak clays. *In situ* testing with a vane or cone penetrometer will help in assessing sampling disturbance effects in gassy or highly structured soils. Approaches such as the SHANSEP technique (stress history and normalized soil engineering properties), see Reference [A.17.4-13], can help provide a more consistent interpretation of standard laboratory tests and will provide history information used to determine the effective overburden stress in normally or underconsolidated clays.

d) Pile length effect

Long piles driven in clay soils can experience capacity degradation due to

1) continued shearing of a particular soil horizon during pile installation,

2) lateral movement of soil away from the pile due to “pile whip” during driving, and/or

3) progressive failure in the soil due to strength reduction with continued displacement (softening).

The occurrence of degradation due to these effects depends on many factors related to both installation conditions and soil behaviour. Methods of estimating the possible magnitude of reduction in capacity of long piles can be found in References [A.17.4-1], [A.17.4-2], [A.17.4-3], [A.17.4-5], [A.17.4-6] and [A.17.4-8].

A.17.4.3.3 Changes in axial capacity in clay with time

Existing axial pile capacity calculation procedures for piles in clay are based on experience tempered by the results of axial pile load tests. In these tests, few of the piles were instrumented and in most cases little or no consideration was given to the effects of time after driving on the development of shear transfer in the soil. Axial capacity of a driven pipe pile in clay computed according to the guidelines set forth in 17.4.1 and 17.4.2 is intended to represent the long-term static capacity of piles in undrained conditions when subjected to axial actions until failure, after dissipation of excess pore water pressure caused by the installation process. Immediately after pile driving, pile capacity in a cohesive deposit can be significantly lower than the ultimate static capacity. Field measurements [A.17.4-10], [A.17.4-14] and [A.17.4-15] have shown that the time required for driven piles to reach ultimate capacity in a cohesive deposit can be relatively long — as much as two to three years. However, it should be noted that the rate of strength gain is highest immediately after driving, and this rate decreases during the dissipation process. Thus a significant strength increase can occur in a relatively short time.

During pile driving in normally to lightly overconsolidated clays, the soil surrounding a pile is significantly disturbed, the stress state is altered, and large excess pore pressures can be generated. After installation, these excess pore pressures begin to dissipate, i.e. the surrounding soil mass begins to consolidate and the pile capacity increases with time. This process is usually referred to as *set-up*. The rate of excess pore pressure dissipation is a function of the coefficient of radial (horizontal) consolidation, pile radius, plug characteristics (plugged versus unplugged pile), and soil layering.

In the case of driven pipe piles supporting a structure where the design actions can be applied to the piles shortly after installation, the time-consolidation characteristics should be considered in pile design. In such cases, the capacity of piles immediately after driving and the expected increase in capacity with time are important design variables that can impact the safety of the foundation system during early stages of the consolidation process.

A number of investigators[A.17.4-16] and [A.17.4-17] have proposed analytical models of pore pressure generation and the subsequent dissipation process for piles in normally consolidated to lightly overconsolidated clays. Since excess pore pressures are generated by pile driving operations, any dissipation of the excess pore pressures after installation should correspond to an equivalent increase in the shear strength of the surrounding soil mass and hence an increase in the capacity of the pile. After dissipation of excess pore pressures, the capacity of a pile approaches long-term capacity, although some strength gain may continue due to secondary processes. In some overconsolidated clays, pile capacity can decrease as pore pressures dissipate, provided the rate of change of radial total stress decreases faster than the rate of change of pore pressure. The analytical models account for the degree of plugging by assuming various degrees of plug formation, ranging from closed- to open-ended pile penetration modes. Input necessary for the analysis includes the soil characteristics (compressibility, stress history, strength, etc.) and the initial site conditions.

In Reference [A.17.4-14] the behaviour of piles subjected to significant axial actions in highly plastic, normally consolidated clays was studied using a large number of model pile tests and some full scale pile load tests. From the study of pore pressure dissipation and load test data at different times after pile driving, empirical correlations were obtained between the degree of consolidation, degree of plugging, and pile shaft shear transfer capacity. The analysis is dependent on the shear strength of the surrounding soil mass. The method is presently limited to use in highly plastic, normally consolidated clays of the type encountered in the Gulf of Mexico, since validation data have been published only for those soils.

In Reference [A.17.4-15], in highly overconsolidated glacial till, capacity was shown to undergo significant short-term reduction associated with pore pressure redistribution and reduction in radial effective stresses during the early stages of the equalization process. The capacity at the end of installation was never fully recovered. Test results for closed-ended steel piles in heavily overconsolidated London clay indicate that there is no significant change in capacity with time[A.17.4-18]. This is contrary to tests on 0,273 m (10,75 in) diameter closed-ended steel piles in overconsolidated Beaumont clay, where considerable and rapid set-up (in four days) was found, see Reference [A.17.4-19].

Caution should be exercised in using the above-mentioned procedures to evaluate set-up, particularly for soils with different plasticity characteristics and under different states of consolidation (especially overconsolidated clays) and piles with D/t ratios greater than 40.

A.17.4.4 Skin friction and end bearing in cohesionless soils

A.17.4.4.1 General

Estimating axial pile capacity in cohesionless soils requires considerable engineering judgment in selecting an appropriate method and associated parameter values. Some of the items that should be considered by geotechnical engineers are detailed in the following subclauses.

A.17.4.4.2 discusses four CPT-based methods for axial pile capacity that incorporate length effects and friction fatigue. Some of these methods have only recently been made available in the literature; they have not yet been frequently compared for routine offshore pile projects. Hence, geotechnical engineering judgment is needed to select the most appropriate method for the design case under consideration. Additional care is required in cases of clay layers at or near pile tip level.

The piles are assumed to be open-ended steel piles of uniform outer diameter. Installation is by impact driving into significant depths of clean siliceous sand. In general, such piles drive unplugged (i.e. they core). However, when they are statically loaded in compression, sufficient inner friction is generally mobilized to cause the pile to act as fully plugged (i.e. the soil plug does not undergo overall slip relative to the pile wall during compression pile loading).

The term *sand* is used hereafter for all cohesionless siliceous soils. Exceptions (e.g. carbonate sands and gravels) are addressed in A.17.4.4.4.

Sands containing calcium carbonate are found extensively in many areas of the oceans. Available data suggest that driven piles in these soils can have substantially lower design strength parameters than those given in Table 17.4-1. Drilled and grouted piles in carbonate sands, however, can have significantly higher capacities than driven piles and have been used successfully in many carbonate areas. The characteristics of carbonate sands are highly variable and local experience should dictate the design parameters selected. For example, available qualitative data suggest that capacity is improved in carbonate soils of high densities and higher quartz contents. Cementation tends to increase end bearing capacity, but results in a loss of lateral pressure and a corresponding decrease in frictional capacity. These materials are discussed further in ISO 19901-4.

The appropriate resistance factors to be used with the methods discussed in A.17.4.4.2 are not provided in A.17.4.4. The designer should carefully evaluate, for each design case, whether the resistance factors provided in 17.3.4 are appropriate or not.

A.17.4.4.2 CPT-based methods for pile capacity

A.17.4.4.2.1 General

In 17.4.4 a simple method for assessing pile capacity in cohesionless soils is presented, which is a modification of methods recommended in API RP2A[A.17.4-20] and previous editions of this recommended practice. Changes were made to remove potential unconservatism. A.17.4.4.2.1 to A.17.4.4.2.5 present recent and more reliable CPT-based methods for predicting pile capacity. These methods are all based on direct correlations of pile unit friction and end bearing data with cone tip resistance values from cone penetration tests (CPT). These CPT-based methods cover a wider range of cohesionless soils, are considered fundamentally better and have shown statistically closer predictions of pile load test results.

Friction and end bearing contributions to pile capacity are assumed to be uncoupled. Hence, for all methods, the representative value of the axial pile capacity in compression ($Q_{\mathrm{r,c}}$) and in tension ($Q_{\mathrm{r,t}}$) of plugged open-ended piles is determined by Equations (A.17.4-2) and (A.17.4-3):

$$Q_{\mathrm{r,c}} = Q_{\mathrm{f,c}} + Q_{\mathrm{p}} = \pi\, D \int f_{\mathrm{c}}(z)\,\mathrm{d}z + q \cdot A_{\mathrm{p}} \tag{A.17.4-2}$$

$$Q_{\mathrm{r,t}} = Q_{\mathrm{f,t}} = \pi\, D \int f_{\mathrm{t}}(z)\,\mathrm{d}z \tag{A.17.4-3}$$

where

$Q_{\mathrm{r,c}}$ is the representative value of the axial pile capacity in compression;

$Q_{\mathrm{r,t}}$ is the representative value of the axial pile capacity in tension;

$Q_{\mathrm{f,c}}$ is the representative value of the total skin friction resistance in compression, in force units;

$Q_{\mathrm{f,t}}$ is the representative value of the total skin friction resistance in tension, in force units;

Q_{p} is the representative value of the end bearing capacity, in force units;

$f_{\mathrm{c}}(z)$ is the unit skin friction in compression, in stress units, which is a function of depth, geometry and soil conditions;

$f_{\mathrm{t}}(z)$ is the unit skin friction in tension, in stress units, which is a function of depth, geometry and soil conditions;

z is the depth below the original sea floor;

q is the unit end bearing at the pile tip, in stress units;

D is the pile outside diameter;

A_{p} is the gross end area of the pile, $A_{\mathrm{p}} = \pi\, D^2/4$.

Since the friction component, Q_f, involves numerical integration, results are sensitive to the depth increment used, particularly for CPT-based methods. As guidance, depth increments for CPT-based methods should be in the order of 1/100 of the pile length (or smaller). In any case, the depth increment should not exceed 0,2 m (0,5 ft).

The four recommended CPT-based methods discussed herein are

- method 1 Simplified ICP-05 (this International Standard),
- method 2 Offshore UWA-05 [A.17.4-21] and [A.17.4-22],
- method 3 Fugro-05 [A.17.4-21] and [A.17.4-23], and
- method 4 NGI-05 [A.17.4-21] and [A.17.4-24].

Method 1 is a simplified version of the design method recommended by Jardine et al.[A.17.4-25], whereas method 2 is a simplified version of the UWA-05 method applicable to offshore pipe piles. Methods 2, 3 and 4 are summarized in Reference [A.17.4-21]. Friction and end-bearing contributions should not be taken from different methods. A general description of methods 1, 2 and 3 is given below, after which details of the various methods are presented separately.

The unit skin friction formulae for open-ended steel pipe piles for CPT-based methods 1, 2 and 3 can all be considered as being special cases of the general formula:

$$f(z) = u\, q_c(z)\left[\frac{\sigma'_{v0}(z)}{p_a}\right]^a A_r^{\,b}\left[\max\left(\frac{L-z}{D}, v\right)\right]^{-c}\left(\tan\delta_{cv}\right)^d\left[\min\left(\frac{L-z}{D}\cdot\frac{1}{v}, 1\right)\right]^e \qquad \text{(A.17.4-4)}$$

where, in addition to prior definitions,

$f(z)$ is the unit skin friction, in stress units, which is a function of depth, geometry and soil conditions;

$q_c(z)$ is the CPT cone-tip resistance at depth, z, in stress units;

$\sigma'_{v0}(z)$ is the effective vertical *in situ* stress of the soil at depth, z;

p_a is the atmospheric pressure, in stress units, Pa = 100 kPa;

A_r is the pile displacement ratio, $A_r = A_w/A_p = 1 - (D_i/D)^2$;

A_w is the area of the rim of the steel pile, $A_w = (\pi/4)\cdot(D^2 - D_i^2)$;

D_i is the pile inside diameter, $D_i = D - 2t$;

t is the pile wall thickness;

L is the embedded length of the pile below the original sea floor;

δ_{cv} is the constant volume friction angle at the interface between the soil and the pile wall.

Recommended values for the parameters, a, b, c, d, e, u and v, for compression and tension are given in Table A.17.4-1.

Table A.17.4-1 — Unit skin friction parameter values for driven open-ended steel piles for methods 1, 2 and 3

Method	Parameter						
	a	b	c	d	e	u	v
Method 1:							
compression	0,1	0,2	0,4	1	0	0,023	$4\sqrt{A_r}$
tension	0,1	0,2	0,4	1	0	0,016	$4\sqrt{A_r}$
Method 2:							
compression	0	0,3	0,5	1	0	0,030	2
tension	0	0,3	0,5	1	0	0,022	2
Method 3:							
compression	0,05	0,45	0,90	0	1	0,043	$2\sqrt{A_r}$
tension	0,15	0,42	0,85	0	0	0,025	$2\sqrt{A_r}$

Additional recommendations for computing unit friction and end bearing of all four CPT-based methods are presented in A.17.4.4.2.2 to A.17.4.4.2.5.

A.17.4.4.2.2 Method 1

a) Friction

Reference [A.17.4-25] presents a comprehensive overview of research work performed at Imperial College on axial pile design criteria of open- and closed-ended piles in clay and sand. The design equations for unit friction in sand in this reference include a soil dilatancy term, implying that unit friction is favourably influenced by soil dilatancy. This influence diminishes with increasing pile diameter. The unit skin friction $f(z)$ of open-ended pipe piles in method 1, given by Equation (A.17.4-4) and the parameter values in Table A.17.4-1, are a conservative approximation of the full calculation in method 1, since dilatancy is ignored and some parameter values were conservatively rounded up/down.

The original "full" design equations in Reference [A.17.4-25] may be used, particularly for small diameter piles [$D < 0{,}76$ m (30 in)], provided that larger resistance factors are considered. See Reference [A.17.4-25] for a discussion on reliability based design using the "full" method 1.

b) End bearing

The unit end bearing, q, for open-ended pipe piles follows the recommendations of Reference [A.17.4-25]. These specify a unit end bearing for plugged piles given by:

$$q = q_{\mathrm{c,av,1,5D}}\left[0{,}5 - 0{,}25\log_{10}\left(D/D_{\mathrm{CPT}}\right)\right] \geqslant 0{,}15\, q_{\mathrm{c,av,1,5D}} \qquad \text{(A.17.4-5)}$$

where, in addition to the general definitions given in A.17.4.4.2.1,

$q_{\mathrm{c,av,1,5D}}$ is the average value of $q_{\mathrm{c}}(z)$ between 1,5 D above the pile tip and 1,5 D below the pile tip

$$q_{\mathrm{c,av,1,5D}} = \left[\int_{L-1{,}5D}^{L+1{,}5D} q_{\mathrm{c}}(z)\,\mathrm{d}z\right] \Big/ (3\,D)$$

D_{CPT} is the diameter of the CPT tool, $D_{\mathrm{CPT}} = 36$ mm for a standard cone with a base area of 10 cm^2.

Reference [A.17.4-25] specifies that plugged pile end bearing capacity applies, which means that the unit end bearing, q, acts across the entire pile tip cross-section, provided both of the following conditions are met:

$$D_\mathrm{i} < 2\ (D_\mathrm{r} - 0{,}3) \tag{A.17.4-6}$$

$$D_\mathrm{i}/D_\mathrm{CPT} < (0{,}083)\ q_\mathrm{c}(z)/p_\mathrm{a} \tag{A.17.4-7}$$

where D_r is the relative density of the sand ($0 \leqslant D_\mathrm{r} \leqslant 1{,}0$).

NOTE Equation (A.17.4-6) is the equation given in Reference [A.17.4-25]. It implies that the maximum pile inside diameter for plugged behaviour in method 1 is 1,4 m. However, experience indicates that offshore piles with appreciably larger inside diameters can still behave as plugged.

If either of the above conditions is not met, then the pile will behave unplugged and Equation (A.17.4-8) should be used for computing the end bearing capacity:

$$Q_\mathrm{p} = \pi\ (D - t)\ t q_\mathrm{c}(z = L) \tag{A.17.4-8}$$

The full pile end bearing computed using Equation (A.17.4-5) for a plugged pile should not be less than the end bearing capacity of an unplugged pile computed according to Equation (A.17.4-8).

A.17.4.4.2.3 Method 2

a) Friction

Reference [A.17.4-21] summarizes the results of recent research work at the University of Western Australia on development of axial pile design criteria of open- and closed-ended piles driven into silica sands. The full design method (described in References [A.17.4-21] and [A.17.4-22]) for unit friction on pipe piles includes a term allowing for favourable effects of soil dilatancy (similar to method 1) and an empirical term allowing for partial soil plugging during pile driving. The authors of Reference [A.17.4-22] recommend for offshore pile design to ignore these two favourable effects (dilatancy and partial plugging), resulting in the recommended Equation (A.17.4-4) and associated Table A.17.4-1. parameter values.

The original "full" design equations in Reference [A.17.4-21] may be used, particularly for small diameter piles [$D < 0{,}76$ m (30 in)], provided that larger resistance factors are considered. See Reference [A.17.4-21] for a discussion on reliability based design using method 2.

b) End bearing

References [A.17.4-21] and [A.17.4-22] present design criteria for representative unit end bearing of plugged open-ended pipe piles. Their "full" design method for pipe piles includes an empirical term allowing for the favourable effect of partial plugging during pile driving. For offshore pile design, References [A.17.4-21] and [A.17.4-22] recommend to ignore this effect, resulting in the recommended design equation for plugged piles in method 2:

$$q = q_\mathrm{c,av,1,5D}\ (0{,}15 + 0{,}45\ A_\mathrm{r}) \tag{A.17.4-9}$$

where, again in addition to the general definitions given in A.17.4.4.2.1,

$q_\mathrm{c,av,1,5D}$ is the average value of $q_\mathrm{c}(z)$ between 1,5 D above the pile tip and 1,5 D below the pile tip

$$q_\mathrm{c,av,1,5D} = \int_{L-1{,}5D}^{L+1{,}5D} q_\mathrm{c}(z)\,\mathrm{d}z \Big/ (3\ D)$$

Since method 2 considers non-plugging under static loading to be exceptional for typical offshore piles, the method does not provide criteria for unplugged piles. The unit end bearing q calculated in Equation (A.17.4-9) is therefore acting across the entire tip cross-section. The use of $q_\mathrm{c,av,1,5D}$ in

Equation (A.17.4-9) is not recommended in sand profiles where the CPT q_c values show significant variations in the vicinity of the pile tip or when penetration into a founding stratum is less than five pile diameters. For these situations, Reference [A.17.4-21] provides guidance on the selection of an appropriate average q_c value for use in place of $q_{c,av,1,5D}$.

A.17.4.4.2.4 Method 3

a) Friction

Method 3 is a modification of method 1 and was developed as part of a research project for API. The unit friction equations were unfortunately misprinted in References [A.17.4-23] and [A.17.4-26] and these references are not to be used in design. However, the correct equations are presented both by Reference [A.17.4-21] and by Equation (A.17.4-4) and the parameter values in Table A.17.4-1. Like the "full" method 1 and the "full" method 2, it is recommended that larger resistance factors are considered when using method 3. See Reference [A.17.4-27], for a discussion on reliability based design using method 3.

b) End bearing

The basis for the representative unit end bearing on pipe piles according to method 3 is presented in the research report to API [A17.4-26] and summarized in Reference [A.17.4-23]. The recommended value of the unit end bearing for plugged piles is given by:

$$q = 8{,}5\ p_a \left(\frac{q_{c,av,1,5D}}{p_a} \right)^{0,5} A_r^{\ 0,25} \tag{A.17.4-10}$$

Both method 2 and method 3 do not specify unplugged end bearing capacity because typical offshore piles behave in a plugged mode during static loading [A.17.4-27]. It can be shown that plugged behaviour applies if either:

— the cumulative thickness of sand layers within a soil plug is in excess of 8 D, or

— the total end bearing Q_p is limited as follows:

$$Q_p \leqslant Q_{f,i,clay} \exp(L_s/D) \tag{A.17.4-11}$$

where in addition to the general definitions in A.17.4.4.2.1

$q_{c,av,1,5D}$ is the average value of $q_c(z)$ between 1,5 D above the pile tip and 1,5 D below the pile tip;

$$q_{c,av,1,5D} = \int_{L-1,5D}^{L+1,5D} q_c(z)\mathrm{d}z \Big/ (3\ D)$$

$Q_{f,i,clay}$ is the representative value of the cumulative skin friction resistance of the clay layers within the soil plug, in force units;

L_s is the length of the plug in the sand layers.

The cumulative frictional capacity of the clay layers within the soil plug ($Q_{f,i,clay}$) can be estimated using similar procedures as for computing estimated pile friction in clay (see 17.4.3).

Equation (A.17.4-11) applies for fully drained behaviour of sand within the pile plug. Criteria for undrained/partially drained sand plug behaviour are presented in Reference [A.17.4-28].

For the exceptional case of unplugged end bearing behaviour in fully drained conditions, reference is made to References [A.17.4-27] and [A.17.4-29] for estimating end bearing capacity.

A.17.4.4.2.5 Method 4

a) Friction

Representative unit skin friction values for tension $f_t(z)$ and compression $f_c(z)$ for driven open-ended steel pipe piles in method 4 are given by Reference [A.17.4-24]:

$$f_t(z) = (z/L)\, p_a\, F_{sig}\, F_{Dr} > 0{,}1\sigma'_{vo} \tag{A.17.4-12}$$

$$f_c(z) = 1{,}3\, (z/L)\, p_a\, F_{sig}\, F_{Dr} > 0{,}1\sigma'_{vo} \tag{A.17.4-13}$$

where, in addition to the general definitions given in A.17.4.4.2.1,

$$F_{sig} = (\sigma'_{vo}/p_a)^{0{,}25} \tag{A.17.4-14}$$

$$F_{Dr} = 2{,}1\, (D_r - 0{,}1)^{1{,}7} \tag{A.17.4-15}$$

$$D_r = 0{,}4\, \ln\{q_c(z)/[22\, (\sigma'_{vo} p_a)^{0{,}5}]\} > 0{,}1 \tag{A17.4-16}$$

Note that values of $D_r > 1$ should be accepted and used.

Like for the "full" methods 1, 2 and 3, higher resistance factors should be considered when using method 4.

b) End bearing

The recommended equation for the representative unit end bearing of plugged open-ended steel pipe piles in method 4 [A.17.4-24] is:

$$q = \frac{0{,}7\, q_{c,av,1,5D}}{1 + 3\, D_r^2} \tag{A.17.4-17}$$

where, once more,

$q_{c,av,1,5D}$ is the average value of $q_c(z)$ between 1,5 D above the pile tip and 1,5 D below the pile tip

$$q_{c,av,1,5D} = \int_{L-1{,}5D}^{L+1{,}5D} q_c(z)\,\mathrm{d}z \Big/ (3\,D)$$

$$D = 0{,}4\, \ln\{q_{c,av,1,5D}/[22\, (\sigma'_{vo} p_a)^{0{,}5}]\} > 0{,}1 \tag{A.17.4-18}$$

Note again that values of $D_r > 1$ should be accepted and used.

The resistance of non-plugging piles should be computed using a unit end bearing value for the steel pile rim [$q_w(z)$] given by Equation (A.17.4-19):

$$q_w(z) = q_c(z) \tag{A.17.4-19}$$

and unit friction [$f_p(z)$] between the soil plug and the inner pile wall given by Equation (A.17.4-20):

$$f_p(z) = 3\, f_c(z) \tag{A.17.4-20}$$

The lower of the plugged resistance, q, of Equation (A.17.4-17) and the unplugged resistance determined by Equations (A.17.4-19) and (A.17.4-20) should be used in design.

A.17.4.4.3 Parameter value assessment

The geotechnical site investigation should provide information that is adequate to capture the spatial variability, horizontally and vertically, of the boundaries and parameter values of all layers.

For any CPT method, the computed pile capacity in sand is most sensitive to cone penetration resistance, q_c, followed by tan δ_{cv} and σ'_{vo}. Since an accurate capacity assessment is a function of the accuracy of both the model and the parameters, guidance regarding selecting appropriate parameter values is given below.

a) Parameter $q_c(z)$

The CPT should measure $q_c(z)$ with apparatus and procedures that are in general accordance with those published by Reference [A.17.4-30]. In particular, Reference [A.17.4-30] prescribes cones with a base area in the range of 500 mm^2 to 2 000 mm^2 and a penetration rate of 20 $\pm$ 5 mm/s.

It should be noted that the CPT-based design methods were established for cone resistance values up to 100 MPa. Caution should be exercised when applying the enclosed methods to sands with higher resistances.

A measured, continuous profile of q_c (z) is preferable to an assumed/interpolated discontinuous profile, but is generally not achievable offshore at large depths below the sea floor with a downhole CPT apparatus. This is generally due to factors such as limited stroke and/or maximum resistance being achieved. When (near) continuous q_c profiles are needed, one can consider overlapping CPT push strokes.

With discontinuous CPT data, a “blocked” q_c profile can be used: the soil profile is divided into layers, in each of which q_c is assumed to vary linearly with depth. “Blocked” profiles should be carefully assessed, particularly when they contain maximum q_c values at the ends of CPT push strokes. When the push strokes contain no maximum q_c data, a moving window may be used to determine the average profile (and its standard deviation), through which a straight line can be fitted. If present, thin layers of weaker material (e.g. silt or clay) need to be modelled conservatively.

For geotechnical investigations where several vertical CPT profiles have been made (e.g. one per platform leg), it is suggested that at least two approaches be employed: capacity should first be based on the combined averaged q_c profile and then based on individual q_c profiles. Judgment is required to select the most appropriate q_c profile and to determine the associated final axial capacity.

b) Parameter σ'_{vo}

Usually, pore water pressures in sands are hydrostatic and in this case σ'_{vo} equals ($\gamma_{sub}z$), where γ_{sub} is the submerged soil unit weight. Offshore sands are generally very dense and often silty. In general, design γ_{sub} values in sands should be based on measured laboratory values (corrected for sampling disturbance effects), which should be compatible with relative density (D_r) estimated from q_c and maximum and minimum dry unit weight values determined in the laboratory.

c) Parameter D_r

Common practice is to use the Ticino sand relationship between q_c and D_r as proposed by Reference [A.17.4-31]:

$$D_r = \frac{1}{2{,}93} \ln\left[\frac{q_c}{205\left(p'_m\right)^{0{,}51}}\right] \tag{A.17.4-21}$$

where

p'_m is the effective mean *in situ* soil stress at depth z, $p'_m = [\sigma'_{v0}(z) + 2\ \sigma'_{h0}(z)]/3$;

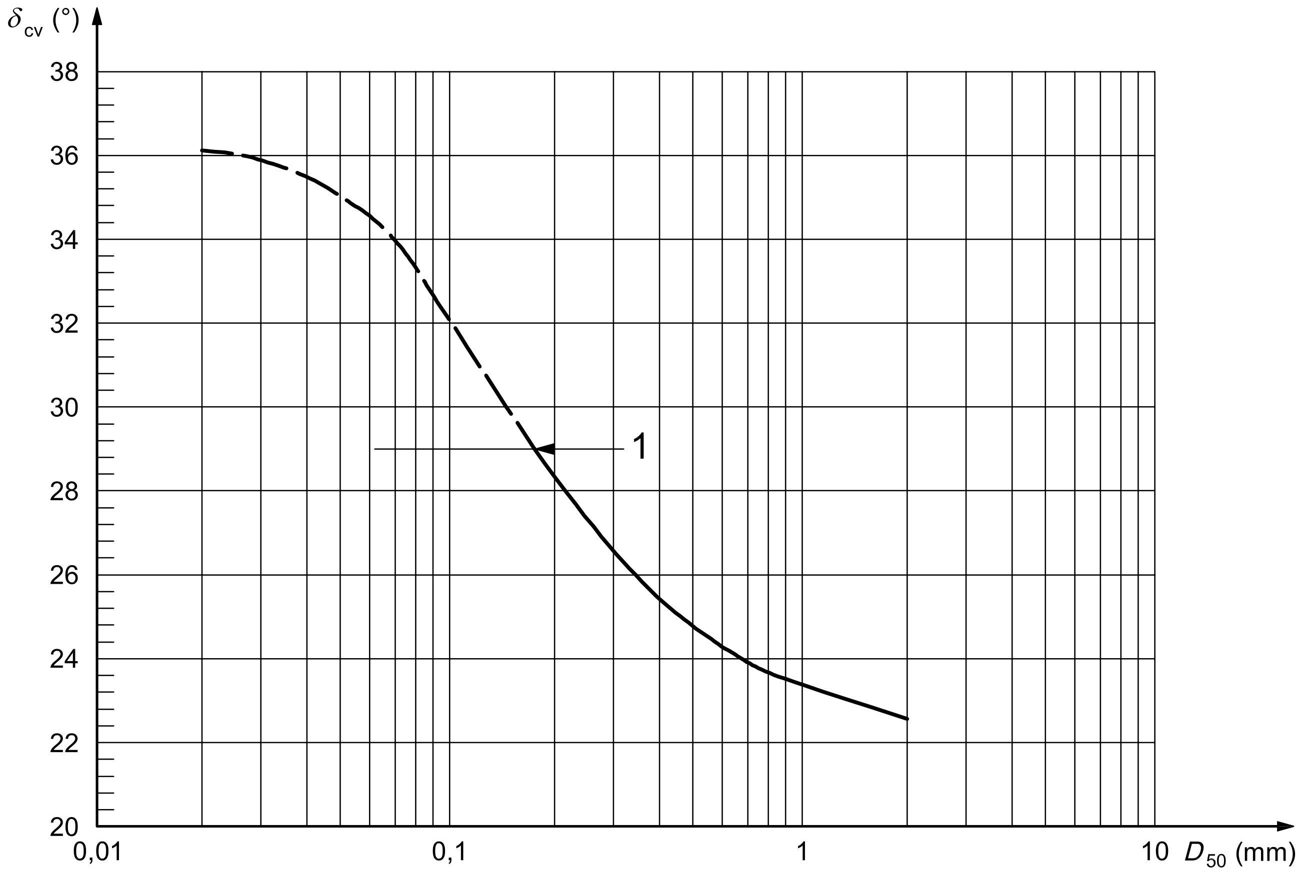
$\sigma'_{h0}(z)$ is the effective horizontal *in situ* stress of the soil at depth z.

Ticino sand is a medium grained silica sand with no fines. A reasonably comprehensive database is available for this sand [A.17.4-32]. However, D_r assessment for method 4 should be according to Equations (A.17.4-16) and (A.17.4-18). Most q_c–D_r relationships are not valid for silty sands. However, q_c may be adjusted for such materials to derive a "clean sand equivalent normalized cone resistance" (see, for example, Reference [A.17.4-33]).

d) Parameter tan δ_{cv}

The constant volume interface friction angle, δ_{cv}, should be measured directly in laboratory interface shear tests. The recommended test method is by ring shear apparatus, but the direct shear box may also be used. Guidance on test procedures is provided in Reference [A.17.4-25].

If site-specific tests cannot be performed, the constant volume interface friction angle may be estimated as a function of mean effective particle diameter (D_{50}) using Reference [A.17.4-25]. An upper limit of tan $\delta_{cv} = 0{,}55$ ($\delta_{cv} = 28{,}8°$) applies to all methods as shown in Figure A.17.4-1. However, for materials with unusually weak grains or compressible structures this method is not always appropriate. Of particular importance are sands containing calcium carbonate, for which specific advice is given in A.17.4.4.4.



Key

1 recommended upper limit tan $\delta_{cv} = 0{,}55$

δ_{cv} interface friction angle

D_{50} mean particle diameter

Figure A.17.4-1— Interface friction angle in sand, δ_{cv}, from direct shear interface tests

A.17.4.4.4 Application of CPT-based methods

Subclauses A.17.4.4.2 and A.17.4.4.3 provide four methods for computing pile capacity in silica sands using CPT data. Points a) to g) below give guidance on the following aspects of pile design using CPT-based methods:

— axial load-displacement behaviour;

— application to soils other than silica sands;

— application to piles with different geometries than typical offshore piles;

— effects of scour on pile capacity.

a) **t–z data for axial shear transfer-displacement response**

No strain softening is applicable. However, unlike for the simplified method given in 17.4.4, the peak unit skin frictions in compression and tension at given depths, $f_c(z)$ and $f_t(z)$, are not unique and both depend on pile geometry. They depend not only on the pile diameter and wall thickness, but also on the total pile penetration. An increased pile penetration will decrease these values at a given depth.

b) **Q–z data for end bearing-displacement response**

Unit end bearing (q) is assumed to be fully mobilized at a pile tip displacement value of 0,1D. This displacement is consistent with the manner in which pile load test data were interpreted.

c) **Application to sands other than siliceous sands**

Other sands include carbonate sands, micaceous sands, glauconitic sands and volcanic sands, silts and clayey sands.

Some cohesionless soils have unusually weak structures/compressible grains. These usually require special *in situ* and/or laboratory tests for selection of an appropriate design method with associated design parameters. See References [A.17.4-34] and [A.17.4-35] for pile design in carbonate sand and Reference [A.17.4-25] for guidelines on pile design in other sands and silts. Consideration should be given to using a design method for clays for cases of low permeability sands and silts. All former methods should be applied cautiously since limited data are available to support their reliability in these sediments.

d) **Cone resistance, $q_c(z)$, in gravel**

The measured q_c data should not be taken at face value in this cohesionless soil type and appropriate adjustments should be made. For example, CPTs made in (coarse) gravels, especially when particle sizes are in excess of 10 % of the CPT cone diameter, are misleading and one possible approach could be to use the lower bound q_c profile. Alternatively, one may estimate an appropriate design q_c profile from adjacent sand layers.

e) **End bearing, Q_p, in presence of weaker clay layers near pile tip**

The $q_c(z)$ data used can have a substantial impact on the fully plugged unit end bearing q. The use of $q_c(z)$ data averaged between 1,5D above the pile tip to 1,5D below the pile tip level should generally be satisfactory, providing $q_c(z)$ does not vary significantly. This is not necessarily the case when clay layers occur. If significant $q_c(z)$ variations occur, then Figure 2.2 of Reference [A.17.4-21] should be used to compute a suitable average $q_{c,av}$ value.

Thin clay layers (less than around 0,1D thick) are problematic, particularly when CPT data are discontinuous vertically and/or not all pile locations have been investigated. Factors to be considered include the variance of layer thickness and of strength and compression parameters. If no direct data are available, a cautious interpretation should be made based on the engineering geology of the surrounding sand soil unit. Offshore piles usually develop only a small percentage of end bearing under extreme conditions. Hence, capacity and settlement calculations, using a finite element model of a pile tip on sand containing weaker layers, may be considered to adequately assess axial pile response under such conditions.

For thick clay layers, shallow geophysical data can be useful to assess layer thickness and elevation. The recommendation in 17.4.4 is to reduce the end bearing component if the pile tip is within a zone up to $\pm 3\,D$ from such layers. If averaging of q_c data is applied to this $\pm 3\,D$ zone, the combined effects can be unduly cautious and such results should be critically reviewed. Similarly, for large diameter piles (say $D > 2$ m), the reduction method in 17.4.4 should be carefully reviewed.

f) Near-shore and onshore piles

In general, for near-shore and onshore piles, the assumptions in A.17.4.4.1 and A.17.4.4.2 are not necessarily valid and should be checked.

Near-shore and onshore pipe piles can respond unplugged when loaded due to insufficient mobilization of inner friction. Similarly, dilatancy effects (which are neglected for offshore piles) may be considered for smaller diameter piles. Scour (especially general scour) can be significant for near-shore pile foundations. In addition, driven closed-ended instead of open-ended steel piles are sometimes used.

The original publications of References [A.17.4-21], [A.17.4-24], [A.17.4-25] and [A.17.4-27] should be consulted for assumptions made and for further guidance; most of these references include methods to provide the capacity of unplugged pipe piles and of closed-ended piles.

g) Scour

Scour (seabed erosion due to wave and current action) can occur around offshore piles. Common types of scour are general scour (overall seabed erosion) and local scour (steep-sided scour pits around single piles or pile groups). There is no generally accepted method to account for scour in axial capacity for offshore piles. Publications such as Reference [A.17.4-36] give techniques for scour depth assessment. In addition, general scour data can be obtained from national authorities.

In lieu of project specific data, A.17.8 gives advice on local scour depth.

Scour decreases axial pile capacity in sand. Both friction and end bearing components are usually affected. This is because scour reduces both q_c and σ'_v (vertical effective stress). For excavations (i.e. general scour), Reference [A.17.4-37] recommends that q_c is simply proportional to σ'_v, i.e.

$$q_{c,f} = \chi \, q_{c,o} \tag{A.17.4-22}$$

where

$q_{c,f}$ is the final q_c value after general scour;

$q_{c,o}$ is the original q_c value before general scour;

χ is the dimensionless scour reduction factor ($\chi = \sigma'_{vf}/\sigma'_{vo}$);

σ'_{vf} is the final vertical effective stress value σ'_v;

σ'_{vo} is the original vertical effective stress value σ'_v.

For large general scour depths and normally consolidated sands, an alternative and conservative approach [A.17.4-38] can be to determine χ from Equation (A.17.4-23):

$$\chi = \frac{1}{1 + 2K_o} \sqrt{\frac{z_S + 2K_o \sqrt{S \cdot z_S + z_S^2}}{S + z_S}} \tag{A.17.4-23}$$

where

S is the general scour depth;

z_S is the depth below the final level of the sea floor ($z_S = z - S$);

K_o is the coefficient of lateral earth pressure (the ratio of the effective horizontal to vertical *in situ* soil stresses, $K_o = \sigma'_{ho}/\sigma'_{vo}$).

A.17.8 gives a σ'_v reduction method for both general and local scour.

A.17.4.5 Skin friction and end bearing of grouted piles in rock

No guidance is offered.

A.17.5 Pile capacity for axial tension

No guidance is offered.

A.17.6 Axial pile performance

A.17.6.1 General

No guidance is offered.

A.17.6.2 Static axial behaviour of piles

An analytical method for determining axial pile performance is provided in Reference [A.17.6-1]. This method makes use of t–z curves of local transfer of axial pile shear, t, against local pile displacement, z, to model the axial support provided by the soil along the side of the pile. An additional Q–z curve is used to model the tip end bearing, Q, against tip displacement, z. Methods for constructing t–z and Q–z curves are given in 17.7.

In some circumstances, i.e. for soils that exhibit strain-softening behaviour and/or where the piles are excessively axially flexible, the actual capacity of the pile can be less than the representative capacity given by Equation (17.4-1). In these cases, an explicit consideration of these effects on axial capacity is warranted.

A.17.6.3 Cyclic axial behaviour of piles

A.17.6.3.1 Qualification

Modelling cyclic effects explicitly can improve the designer's insight into the relative importance of the cyclic characteristics of the actions. On the other hand, extreme care should be exercised in applying this approach; historically, cyclic effects have been taken into account implicitly rather than explicitly. Design methods developed and calibrated on an implicit basis generally need extensive modification where explicit algorithms are employed.

A.17.6.3.2 Actions

Axial actions on piles are developed from a wide variety of operating, structural and environmental sources. Permanent and variable actions are generally long duration actions and are often referred to as static actions. Environmental actions are developed by winds, waves and currents, earthquakes and ice floes. These actions can have both low and high frequency cyclic components in which the rates of change of actions and action durations are measured in seconds. Storm and ice can cause several thousand cycles of (relatively speaking) low frequency actions, while earthquakes can induce several tens of cycles of high frequency actions[A.17.6-2].

A.17.6.3.3 Static capacity

For most fixed offshore structures supported on piles, experience has proven the adequacy of determining pile penetration based on static capacity evaluations, with static design actions and commonly accepted working stress design (WSD) factors of safety that, in part, account for the cyclic effects[A.17.6-1]. The partial action and resistance factors for pile design have been calibrated against these factors of safety.

Detailed consideration of cyclic effects can be warranted when there are unusual limitations on pile penetrations or when certain soils, conditions related to actions or novel structures (e.g. compliant towers) are involved.

A.17.6.3.4 Cyclic effects

Compared with long-term static actions, cyclic actions can have the following important influence on pile axial capacity and stiffness. They can

— decrease capacity and stiffness due to repeated actions[A.17.6-3], or

— increase capacity and stiffness due to high rates of change of actions[A.17.6-4].

The resultant effect on capacity is primarily influenced by the pile properties (stiffness, length, diameter, material), the soil characteristics (type, stress history, strain rate and cyclic degradation) and the action characteristics (numbers and magnitudes of repeated actions). Cyclic actions can also cause accumulation of pile displacements and either stiffening and strengthening or softening and weakening of the soils around the pile. Hysteretic and radiation damping dissipate the energy provided by the actions in the soil[A.17.6-2]. For earthquakes, the free-field ground motions (independent of the presence of the piles and structure) can develop important cyclic straining effects in the soils; these effects can influence pile capacity and stiffness[A.17.6-5].

A.17.6.3.5 Analytical models

A variety of analytical models have been developed and applied to determine the cyclic axial behaviour of piles. These models can be grouped into two general categories, as below.

a) Discrete element models

The soil around the pile is idealized as a series of uncoupled “springs” or elements attached between the pile and the far field soil (usually assumed rigid). The material behaviour of these elements can vary from linearly elastic to non-linear, hysteretic and rate dependent. The soil elements are commonly referred to as t–z (shaft resistance-displacement) and Q–z (tip resistance-displacement) elements, see References [A.17.6-6] to [A.17.6-8]. Linear or non-linear dashpots (velocity dependent resistances) can be placed in parallel and in series with the discrete elements to model radiation damping and rate of change of action effects[A.17.6-9]. The pile can also be modelled as a series of discrete elements, e.g. rigid masses interconnected by springs, or modelled as a continuous rod, either linear or non-linear. In these models, material properties (soil and pile) can vary along the pile.

b) Continuum models

The soil around the pile is idealized as a continuum attached continuously to the pile. The material behaviour can incorporate virtually any reasonable stress-strain rules the analyst can devise. Depending on the degree of non-linearity and heterogeneity, this model can be quite complicated. Again, the pile is typically modelled as a continuous rod, either linear or non-linear. In these models material properties can vary in any direction, see References [A.17.6-10] and [A.17.6-11].

There is a wide range of assumptions that can be used regarding boundary conditions, solution characteristics, etc. which lead to an unlimited number of variations for either of the two approaches.

Once the idealized model is established and the relevant equations are developed, then a solution technique should be selected. For simple models, a closed form analytical approach is sometimes possible. Otherwise, a numerical procedure should be used. In some cases, a combination of numerical and analytical approaches is helpful. The most frequently used numerical solution techniques are the finite difference method and the finite element method. Either approach can be applied to both the discrete element and continuum element models. Discrete element and continuum element models are occasionally combined[A.17.6-2] and [A.17.6-9]. Classical finite element models have been used for specialized analyses of piles subjected to monotonic axial actions[A.17.6-10].

For practical reasons, discrete element models, solved numerically, have seen the most use in evaluation of piles subjected to high intensity cyclic action. Results from these models are used to develop information on pile accumulated displacements and on pile capacity following high intensity cyclic actions, see References [A.17.6-7] and [A.17.6-8].

Elastic continuum models solved analytically (similar to those used in machine vibration analyses) have proven to be useful for evaluations of piles subjected to low intensity, high frequency cyclic actions at or below design working levels[A.17.6-10] and [A.17.6-11]. At higher intensity actions, where material behaviour is likely to be non-linear, the continuum model solved analytically can still be used by employing equivalent linear properties that approximate the non-linear, hysteretic effects[A.17.6-12].

A.17.6.3.6 Soil characteristics

A key part of developing realistic analytical models to evaluate cyclic effects on piles is the characterization of soil-pile interaction behaviour. High quality *in situ*, laboratory and model-prototype pile load tests are essential in such characterizations. In developing (soil) characterizations relevant for soil-pile interaction, it is important that pile installation and relevant conditions of the actions on a pile be integrated into the testing programmes[A.17.6-2] and [A.17.6-8].

In situ tests (e.g. vane shear, cone penetrometer, pressuremeter) can provide important insights into in-place soil behaviour and stress-strain properties[A.17.6-13]. Both low and high amplitude stress-strain properties can be developed. Long-term (static, creep), short-term (dynamic, impulsive) and cyclic (repeated) actions sometimes can be simulated with *in situ* testing equipment.

Laboratory tests on representative soil samples permit a wide variety of stress-strain conditions to be simulated and evaluated[A.17.6-14]. Soil samples can be modified to simulate pile installation effects (e.g. remoulding and reconsolidating to estimated *in situ* stresses). The samples can be subjected to different boundary conditions (triaxial, simple-shear, interface shear) and to different levels of sustained and cyclic shear time histories to simulate in-place conditions of applied actions.

Tests on model and prototype piles are another important source of data for developing soil characterizations for cyclic analyses. Model piles can be highly instrumented and repeated tests can be performed in soils and for a variety of actions[A.17.6-8] and [A.17.6-15]. Geometrical scale, time scale and other modelling effects should be carefully considered in applying results from model tests to analyses of prototype behaviour.

Data from load tests on prototype piles are useful for calibrating analytical models[A.17.6-16] to [A.17.6-19]. Such tests, even if not highly instrumented, can provide data to guide development of analytical models. These tests can also provide data for verifying results of soil characterizations and analytical models, as shown in References [A.17.6-2], [A.17.6-8], [A.17.6-9], [A.17.6-20] and [A.17.6-21]. Prototype pile load testing coupled with *in situ* and laboratory soil testing and realistic analytical models can provide an essential framework for making realistic evaluations of the responses of piles to cyclic axial actions.

A.17.6.3.7 Analysis procedure

The primary considerations in performing an analysis of cyclic axial effects on a pile using discrete element models are as below.

a) Actions

The actions on the pile head should be characterized in terms of their magnitudes, durations and numbers of cycles. This includes both long-term actions and short-term cyclic actions. Typically, the design static and cyclic actions expected during a design event are chosen.

b) Pile properties

The properties of the pile including its diameter, wall thickness, stiffness, weight and length should be defined. This will require an initial estimate of the pile penetration that is appropriate for the design actions. Empirical, pseudo-static methods based on pile load tests or soil tests can be used to make such estimates.

c) Soil properties

Different analytical approaches will require different soil parameters. For the continuum model, the elastic and damping properties of the soil are required. In the discrete element model, soil resistance–displacement relationships along the pile shaft t–z and at its tip Q–z should be determined. *In situ* and laboratory soil tests and model and prototype pile load tests can provide a basis for such determinations. These tests should at least implicitly include the effects of pile installation, types of actions and time scales. In addition, the test should be performed so as to provide insight regarding the effects of the characteristics of the actions on the pile. Most importantly, the soil behaviour characteristics should be appropriate for the analytical model(s) used, duly recognizing the empirical bases of these models.

d) Cyclic analyses

Analyses should be performed to determine the response (resistance and displacement) characteristics of the pile subjected to its design static and cyclic actions. Recognizing the inherent uncertainties in evaluations of pile actions and soil-pile behaviour, parametric analyses should be performed to evaluate the sensitivities of the pile response to these uncertainties. The analytical results should develop realistic predictions of pile resistance and accumulated displacements for design actions. In addition, following the simulation of static and cyclic design actions, the pile should be further analysed so as to estimate its reserve capacity.

A.17.6.3.8 Performance requirements

A primary objective of these analyses is to ensure that the pile and its penetration are adequate to meet the structure's requirements.

In conventional static capacity based design, the pile design actions (factored permanent and variable actions plus factored extreme environmental actions) are compared against the factored pile capacity. The factored actions are defined in Clause 9. The pile capacity is defined as the integrated shaft and tip resistance (17.4 and 17.5). This procedure ensures that the pile has an adequate reserve above the design actions in order to accommodate uncertainties in actions and pile resistances.

The pile performance for explicit cyclic analyses should be evaluated separately. The pile should have a capacity that provides an adequate margin of reserve above the design actions. In addition, the pile should not settle or pull-out, nor accumulate displacements to the extent that could constitute failure of the structure-foundation system.

A.17.6.4 Overall axial behaviour of piles

No guidance is offered.

A.17.7 Soil reaction for piles under axial compression

A.17.7.1 General

No guidance is offered.

A.17.7.2 Axial shear transfer t–z curves

Theoretical curves may be constructed in accordance with Reference [A.17.7-1]. Empirical t–z curves based on the results of model and full-scale pile load tests may follow the procedures for clay soils described in Reference [A.17.7-2].

A.17.7.3 End bearing resistance–displacement Q–z curves

No guidance is offered.

A.17.8 Soil reaction for piles under lateral actions

Generally, under lateral actions, clay soils behave as a plastic material which makes it necessary to relate pile-soil deformation to soil resistance. To facilitate this procedure, lateral soil resistance-displacement p–y curves should be constructed using stress-strain data from laboratory soil samples. The ordinate for these curves is soil resistance p and the abscissa is pile wall displacement y. By iterative procedures, a compatible set of lateral resistance-displacement values for the pile-soil system can be developed.

For a more detailed study of the construction of p–y curves, see Reference [A.17.8-1] for soft clay, Reference [A.17.8-2] for stiff clay, Reference [A.17.8-3] for sand and Reference [A.17.8-4] for layered soils.

Scour (seabed sediment erosion due to wave and current action) can occur around offshore piles. Scour reduces lateral soil support, leading to an increase in pile maximum bending stress. Scour is generally not a problem for cohesive soils, but should be considered for cohesionless soils. Common types of scour are

a) general scour (overall seabed erosion), and

b) local scour (steep sided scour pits around single piles).

Publications such as Reference [A.17.8-5] give techniques for scour depth assessment. In addition, general scour data may be obtained from national authorities. In the absence of project specific data, for an isolated pile a local scour depth equal to 1,5 D and an overburden reduction depth equal to 6 D may be adopted, D being the pile outside diameter; see Figure A.17.8-1.

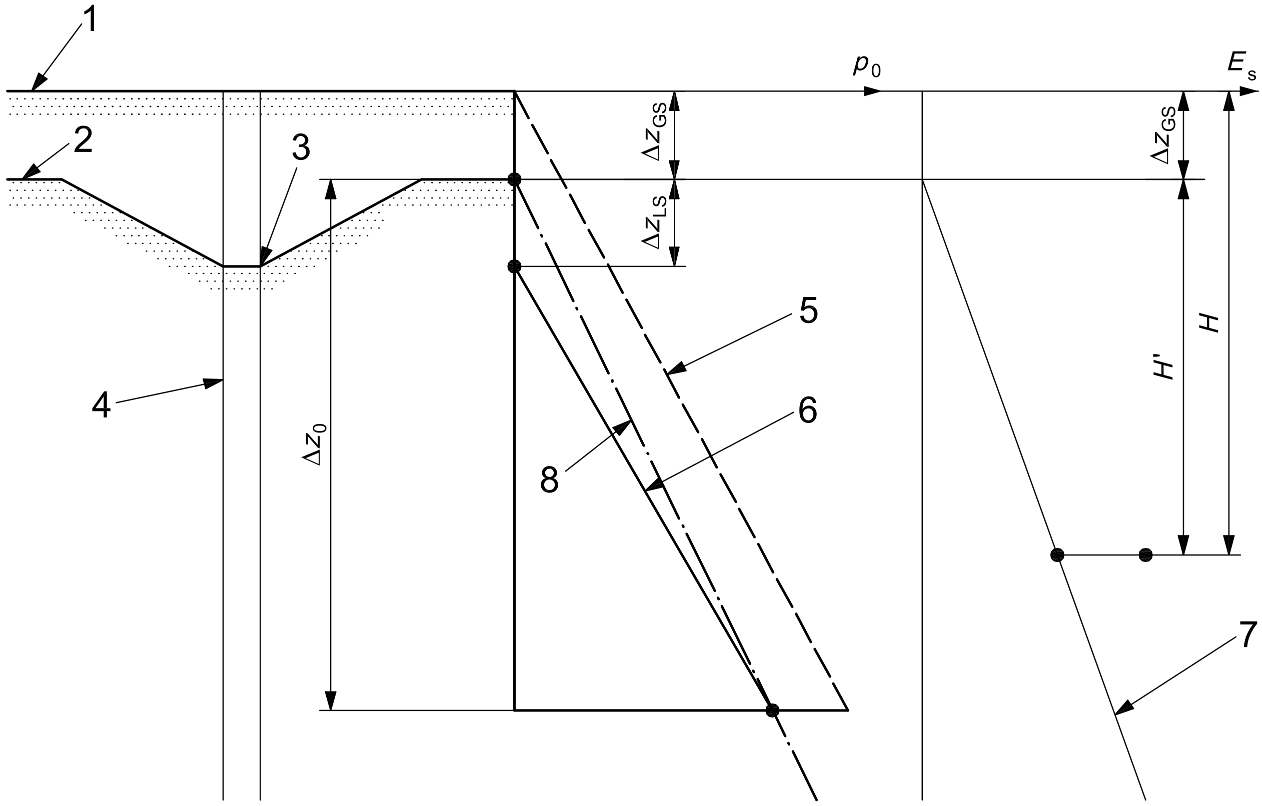
Reduction in lateral soil support is due to two effects:

— a lower ultimate lateral pressure caused by decreased vertical effective stress p_0;

— a decreased initial modulus of subgrade reaction modulus (E_S).

There is no general accepted method to allow for scour in the p–y curves for offshore piles. Figure A.17.8-1 suggests one of the methods for evaluating p_0 and E_S as a function of scour depths. In this method, general scour reduces the p_0 profile uniformly with depth, whereas local scour reduces p_0 linearly with depth to a certain depth below the base of the scour pit. Subgrade modulus reaction values (E_S) may be computed assuming the general scour condition only.

Other methods, based upon local practice and/or experience, may be used instead.



Key

1	original sea floor level
2	level after general scour
3	level of local scour
4	pile
5	no scour case
6	local scour case
7	$E_s = kH'$, where k is given in Table 17.8-3
8	general scour only
Δz_{GS}	general scour depth
Δz_{LS}	local scour depth (1,5 × Δ typical)
Δz_0	overburden reduction depth (6,0 × Δ typical)
p_0	vertical effective stress
E_s	initial modulus of subgrade reaction
H	depth below original sea floor
H'	depth below final general sea floor

Figure A.17.8-1 — p–y lateral support — Scour model

A.17.9 Pile group behaviour

A.17.9.1 General

Routine numerical analysis of pile groups can be divided into two main categories.

The first category, which is computationally the simplest, uses algebraic expressions to define the elastic single pile resistance to general (axial, lateral and torsional) actions[A.17.9.1]. The group resistance is determined by modifying the single pile expressions to account for elastic pile-soil-pile interaction.

The second analysis category, which is normally performed for offshore pile groups, is more rigorous. Methods are usually hybrid, employing a mixture of discrete p–y curves (Winkler approach) and continuum soil behaviour, first described in Reference [A.17.9-2] for lateral analysis. Since then, numerous programs have been developed worldwide for general types of action. Typically, the non-linear single pile resistances to general actions are computed using axial t–z and lateral p–y curves and combined with elastic interaction expressions similar to the first category. The resulting equations are solved for various pile head fixity conditions and/or pile cap restraint to determine the non-linear group resistance and individual pile forces and moments, plus the so-called "z- and y-modifiers".

For more detailed discussions see References [A.17.9-1] and [A.17.9-3].

A.17.9.2 Axial behaviour

In general, group effects depend considerably on pile group geometry and penetrations and thickness of any bearing stratum underneath the pile tips[A.17.9-1] and [A.17.9-3].

A.17.9.3 Lateral behaviour

Experience confirms that the available tools for analysis of pile groups subjected to lateral actions provide approximate answers that sometimes deviate significantly from observed behaviour, particularly with regard to displacement calculations. Also, limitations in site investigation procedures and in the ability to predict soil-pile interaction behaviour for a single pile produce uncertainty regarding proper soil input to group analyses. Therefore, multiple analyses should be performed for pile groups using two or more methods of analysis and upper-bound and lower-bound values of soil properties in the analyses. By performing such analyses, the designer will obtain an appreciation for the uncertainty involved in his predictions of foundation performance and can make more informed decisions regarding the structural design of the foundation and structure elements.

A.17.9.4 Pile group stiffness and structure dynamics

Insight regarding how changes in foundation stiffness can impact the natural frequencies of tall steel structures is provided in Reference [A.17.9-4].

A.17.9.5 Resistance factors

No guidance is offered.

A.17.10 Pile wall thickness

A.17.10.1 General

No guidance is offered.

A.17.10.2 Pile stresses

No guidance is offered.

A.17.10.3 Pile design checks

No guidance is offered.

A.17.10.4 Check for load case due to weight of hammer during hammer placement

Designers should use the K factor that is appropriate for the cantilevered end-span of a continuous beam, which differs from the theoretical or commonly recommended K value for a fixed-end cantilever. Values of effective length factors, K, of 2,3 and 2,4 for pile and conductor add-ons, respectively, are suggested for typical installations.

In the beam column resistance check, for either nearly vertical or inclined add-on sections, the recommended method of calculating the secondary P - Δ moments due to first-order lateral deflections is more accurate[A.17.10-1] than the familiar procedure of modifying the primary support moment (M) by means of the moment reduction factor C_m as discussed in Clause 13.

A.17.10.5 Stresses during driving

No guidance is offered.

A.17.10.6 Minimum wall thickness

No guidance is offered.

A.17.10.7 Allowance for underdrive and overdrive

No guidance is offered.

A.17.10.8 Driving shoe

The designer should be aware that pile buckling and pile refusal incidents in very dense sands have been associated with the use of external chamfers at the pile tip. Although factors other than the shape of the pile tip contribute to buckling, the use of an external chamfer can increase the potential for buckling and/or refusal.

A.17.10.9 Driving head

No guidance is offered.

A.17.11 Length of pile sections

No guidance is offered.

A.17.12 Shallow foundations

No guidance is offered.

A.18 Corrosion control

Guidance for coatings is given in References [A.18.4-1] to [A.18.4-4].

Applicable cathodic protection codes are given in References [A.18.4-5] to [A.18.4-12].

A.19 Materials

A.19.1 General

No guidance is offered.

A.19.2 Design philosophy

A.19.2.1 Material characterization

Key characteristics of structural steel are as follows.

a) Strength

Strength is the primary parameter for structural steel and the yield strength is used throughout Clauses 13 to 16 to determine the resistance of structural components. The strength is measured by tensile testing of specimens. Fatigue resistance of as-welded structures (Clause 16) tends to fall in the same scatter band, regardless of strength, so increasing design stresses for high strength steel tends to shorten fatigue lives.

b) Toughness

Toughness becomes more important as the magnitudes of varying actions increase and as service temperatures decrease. While fracture control is based on preventing the initiation of cracks, notch toughness of the steel affects both terminal fatigue condition and early fracture resistance. Toughness can be measured or controlled by

- controlling the chemistry and processing of the steel,
- Charpy V-notch (CVN) testing, or
- crack tip opening displacement (CTOD) testing.

Both Charpy testing and CTOD testing are undertaken at temperatures related to the lowest temperature to which a structure is expected to be exposed.

A.19.2.2 Material selection criteria

A.19.2.2.1 Yield strength requirements

No guidance is offered.

A.19.2.2.2 Structure exposure level

No guidance is offered.

A.19.2.2.3 Component criticality

Connections whose failure would endanger the entire structure should receive detailed consideration, particularly if they are subject to factors known to enhance the risk of fracture. Such factors include conditions of high restraint (e.g. adverse geometry, multi-axial stresses, thick sections), high residual stresses arising from fabrication, through-thickness shrinkage strains after welding, material properties and the possibility of hydrogen pickup.

Practice will vary from one region to another, but for more severe environments such as the North Sea consideration should be given to the notch toughness and its variation with temperature. In some areas, the structural steel used for construction of a fixed steel structure will generally be the same above and below water and will be selected with reference to the most severe anticipated conditions. Applications such as subsea templates and other structures where the steel is completely and permanently submerged may use less stringent temperature requirements for notch toughness. Where high strength steels have been utilized in fatigue conditions, the level of cathodic protection provided should be taken into account.

Particularly critical joints can benefit from the use of tougher class CV2 steels.

Those structures continuously above the water-line, or only intermittently submerged in the splash zone, usually experience lower temperatures in winter and can additionally suffer shock loading from causes such as wave slam and boat impact. For these structures and any other critical structures or connections where it is recognized that a significant risk of brittle fracture exists, the tougher steels of class CV2 should be considered. Special attention should be given to welding procedures for higher-strength steels of group IV and above [above 460 MPa (67 ksi)].

For critical joints liable to the action of any of these factors, active consideration should be given to the use of CV2Z and CV2ZX quality steel having improved through-thickness (Z direction) properties as well as high toughness, including the possible specification of crack arrest and CTOD testing. The use of post weld heat treatment (PWHT) for these joints may be considered as another alternative. Although the sensitivity of structural steels to the factors noted above generally increases with yield strength, this tendency does not necessarily preclude the use of high-strength steels, as the processing route can have considerable consequences for the mechanical behaviour.

Particular consideration should be given to through-thickness behaviour under constrained loadings. Although the brace ends at tubular connections are also subject to stress concentration, the conditions of service are not quite as severe as for node cans. For critical braces, for which brittle fracture would be catastrophic, consideration should be given to the use of stub-ends in the braces having the same class as the node can, or one class lower. This recommendation need not apply to the body of braces (between joints).

A.19.2.2.4 Lowest anticipated service temperature

The bulk of an offshore structure is submerged in sea water. Except for arctic structures, temperatures are above freezing, with the minimum temperature in the range of – 2 °C to 10 °C (28 °F to 50 °F), depending on geographical location.

The values of the LAST presented in Table A.19.2-1 have been suggested for various offshore operating areas. For the Gulf of Mexico, a somewhat higher temperature of 21 °C (70 °F) prevails for both air and water during the hurricane season, which corresponds to the occurrence of significant actions approaching design level. At depths below 300 m (1 000 ft), the Gulf of Mexico water temperature is 4 °C (40 °F), like deep oceans worldwide.

Table A.19.2-1 — Recommended lowest anticipated service temperatures (LAST)

Location	LAST in air	LAST in water
Gulf of Mexico	– 10 °C (+ 14 °F)	+ 10 °C (+ 50 °F)
Southern California	0 °C (+ 32 °F)	+ 4 °C (+ 40 °F)
Cook Inlet, Alaska	– 29 °C (– 20 °F)	– 2 °C (+ 28 °F)
North Sea, south of Latitude 62°	– 10 °C (+ 14 °F)	+ 4 °C (+ 40 °F)
North Sea, north of Latitude 62°	Site-specific data should be used	
Mediterranean Sea, north of Latitude 38°	– 5 °C (+ 23 °F)	+ 5 °C (+ 41 °F)
Mediterranean Sea, south of Latitude 38°	0 °C (+ 32 °F)	+ 10 °C (+ 50 °F)

A.19.2.2.5 Other considerations

Resistance to lamellar tearing is an important property for steels strained normal to the plane of rolling (i.e. parallel to the thickness). The formation of planar discontinuities during the steel manufacturing process creates potential sites for steel separation if tensile actions are applied in the through-thickness direction.

Weldability is a measure of the steel’s resistance to cracking due to hydrogen and other embrittlement mechanisms in the HAZ immediately adjacent to a weld, using normal welding techniques. The effects of welding can cause modifications to the microstructure within the steel adjacent to the weld; consequently, achieving the toughness requirements in the HAZ can be affected by the steel selection as well as by the welding processes and parameters.

A.19.2.3 Selection process

No guidance is offered.

A.19.2.4 Material category approach

The MC method has evolved from practices in the Gulf of Mexico and other worldwide applications where ASTM, API and AWS standards are used. Each structure to be designed and built is assigned to a particular category. MC3 includes small structures in water depths less than 130 m (400 ft) which are unmanned during the design environmental event. MC2 refers to larger structures in deeper water where failure consequences and fatigue loading become more significant. MC1 is reserved for structures with very high failure consequences in terms of monetary loss, wasted natural resources, pollution and/or loss of human life. These definitions are closely aligned with the exposure levels L3, L2, L1 discussed in Clause 6.6.

A.19.2.5 Design class approach

The DC method has evolved from North Sea practices, consistent with the use of BS, EEMUA, NORSOK and EN standards. In particular, the DC approach is used for large integrated engineering procurement installation and construction (EPIC) projects, where considerable resources are often devoted to material selection.

A.19.3 Strength groups

A.19.3.1 General

The steel strength groupings also provide for distinct categories of CVN impact toughness. The impact energy and test temperature required for each class varies depending on the steel strength, details of the application, the steel specification and the location of the structure (affecting the LAST). Care should be taken to ensure that the impact toughness level is appropriate for each intended application. The impact toughness should be in accordance with the specific steel specification.

A.19.3.2 Group I steels

Group I designates mild steels with a specified minimum yield strength (SMYS) of 275 MPa (40 ksi) or less. Routine weldability can generally be expected as long as the value of the carbon equivalent (P_{CE}), computed from Equation (A.19.3-1), is less than or equal to 0,45:

$$P_{\mathrm{CE}} = P_{\mathrm{C}} + P_{\mathrm{Mn}}/6 + (P_{\mathrm{Cr}} + P_{\mathrm{Mo}} + P_{\mathrm{V}})/5 + (P_{\mathrm{Ni}} + P_{\mathrm{Cu}})/15 \qquad \text{(A.19.3-1)}$$

where

P_{C} is the weight percentage of carbon;

P_{Mn} is the weight percentage of manganese;

P_{Cr} is the weight percentage of chromium;

P_{Mo} is the weight percentage of molybdenum;

P_{V} is the weight percentage of vanadium;

P_{Ni} is the weight percentage of nickel;

P_{Cu} is the weight percentage of copper.

A.19.3.3 Group II steels

Group II designates intermediate strength steels with the SMYS of 280 MPa (41 ksi) to 395 MPa (57 ksi). Their weldability should be assured by requiring approved weldability data from the supplier, normally by testing in accordance with a recognized international standard, or by limiting the carbon equivalent of the steel depending on toughness class.

P_{CE} calculated as in Equation (A.19.3-1) should not exceed 0,43 for all toughness categories (except for NT steels, where P_{CE} may be 0,45).

P_{CM} (carbon equivalent parameter for cracks, modified), computed from Equation (A.19.3-2), should not exceed 0,23 for all toughness categories. This requirement does not apply to NT steels.

$$P_{CM} = P_C + P_{Si}/30 + (P_{Mn} + P_{Cu} + P_{Cr})/20 + P_{Ni}/60 + P_{Mo}/15 + P_V/10 + 5\,P_B \qquad \text{(A.19.3-2)}$$

where, in addition to the symbols given in A.19.3.2,

P_{Si} is the weight percentage of silicon;

P_B is the weight percentage of boron.

A.19.3.4 Group III steels

Group III designates high strength steels with the SMYS in the range of 400 MP to 455 MPa (58 ksi to 66 ksi). These steels, when supplied in plate form, are usually manufactured by either quenched and tempered (Q&T) or thermo-mechanically controlled process (TMCP) methods. They share the weldability characteristics of group IV.

For group III and above, the following should be considered:

a) weldability — some modern, low P_{CE} (low P_{CM}) plates have lean compositions and may be more readily weldable than a lower grade/lower strength plate that has been made to a higher carbon composition and normalized;

b) fatigue problems, including corrosion fatigue, which can result from the use of higher working stresses — consequently the higher tensile strengths of these steels is sometimes not exploited in such situations;

c) notch toughness in relation to other elements of fracture control, such as fabrication practices, inspection procedures and the service environment, including corrosion and hydrogen embrittlement, whether from cathodic protection or microbiological origins, the effects of temperature and the effects of high rates of loading.

A.19.3.5 Group IV steels

Strength groups IV and V have been introduced to accommodate higher strength structural steels (beyond current usage) when available. Specific strength levels for these shall be designated by the designer. The designer should take into account any difficulties of matching the yield strength of the weld metal to those of the steels, and factors such as thickness and shrinkage stresses should be considered when specifying welding processes.

Group IV designates high-strength steels with the SMYS between 460 MPa (67 ksi) and 500 MPa (72 ksi), which can be supplied in the Q&T condition or produced by TMCP. Such steels are usually characterized by a relatively lean chemistry and good weldability and possess high fracture toughness in addition to high strength. Normalized steel at this strength generally cannot provide the combination of tensile properties and fracture toughness in conjunction with good weldability, and so the normalized condition is not specified for class CV2.

Most of the experience to date has been accumulated for the Q&T condition. The user should take steps to ensure that the specified minimum yield strength is clearly defined for both Q&T and TMCP steels at all thicknesses. The user is reminded of the tendency for steels to be more susceptible to hydrogen embrittlement at higher yield strength. Hydrogen can arise from a number of sources, including excessive cathodic protection of submerged structures and the action of sulphate-reducing bacteria under anaerobic conditions. Some TMCP steels with SMYS in the range 450 MPa to 480 MPa have been resistant to hydrogen attack in pipeline service. TMCP steel with 420 MPa SMYS is being used in fracture-critical bridge members. Conventional high strength steels that can be hardened by use of higher alloy content are likely to be more susceptible to hydrogen damage.

A.19.3.6 Group V steels

Group V are steels having an SMYS above 500 MPa (72 ksi) together with high levels of impact toughness and weldability. Experience to date is very limited, see Reference [A.19.2-1].

A.19.4 Toughness classes

No guidance is offered.

A.19.5 Applicable steels

Listed steels in Annexes C and D generally have an acceptable track record of weldability. When specifying or allowing unlisted steels, weldability and suitability should be assessed.

Offshore grade steel castings are most readily obtained in strength levels equivalent to Grade 355 (50 ksi), but higher strength castings of 540 MPa (78 ksi) and 690 MPa (100 ksi) are also obtainable, albeit in more limited size ranges. Examples of the use of castings include

a) cast nodes for fatigue sensitive area of fixed steel structures,

b) J-tube trumpet sections,

c) complex nodes for topsides modules or integrated decks,

d) padeye or trunnion (padear) assemblies, and

e) spreader bar ends.

Castings can offer solutions to specific problem areas, such as the avoidance of massive welding in the fabrication of trunnions and improved transitions between heavy and light sections or between tubular members and open section steelwork. Reference [A.19.5-1] contains information on steel castings for offshore structures.

A.19.6 Cement grout for pile-to-sleeve connections and grouted repairs

A.19.6.1 Grout materials

No guidance is offered.

A.19.6.2 Onshore grout trial

75 mm (3 in) cubes have been used for a large majority of the grout compressive strength determination of laboratory test specimens used to develop the connection capacity formulation. Therefore, the specified grout compressive strength, f_{cu}, used in the capacity equations given in 15.1 should be based on, or corrected to, a 75 mm cube.

Tests reported [A.19.6-1] show that cube conversion factors are dependent on the grout age at testing as well as the size. This is probably due to the different rates of increase in strength and elastic moduli properties observed in grouts. Therefore, care is required in obtaining appropriate conversion factors.

The volume of grout mixed and the number of samples tested at each age should be sufficiently large to be representative of the production grouting.

A.19.6.3 Offshore grout trial

The offshore grouting process is unusual in that only one chance is available to achieve a satisfactory connection. Therefore, it is imperative that all measures possible be taken to ensure that the grout and grouting equipment used in the process are satisfactory prior to pumping the grout. The offshore trial provides a functional check on the equipment, including the calibration of gauges and material compliance. The volume of grout mixed and the number of samples tested at 30 h should be sufficiently large to be representative of the production grouting.

A.19.6.4 Offshore quality control

The measurements of the specific gravity of the grout slurry prior to grouting and on the return through the annulus, are the only quantitative quality control methods available at the time of grouting. Therefore, it is important that these measurements be taken frequently and accurately to ensure, as far as is reasonably practical, that the in-place grout achieves its strength compliance.

In cement and water grouts, the specific gravity is an accurate measure of the mix proportions. However, when fillers and admixtures are included in the mix, additional care is required to control batching and dosing. If prebatched bulk cement-fillers-admixtures are used, care should be taken to ensure that segregation of the constituent materials does not occur during transportation, handling and storage.

ASTM[A.19.6-2] is a suitable standard for casting and testing grout cubes.

A.20 Welding, fabrication and weld inspection

A.20.1 General

No guidance is offered.

A.20.2 Welding

A.20.2.1 Selected generic welding and fabrication standards

The standard(s) selected by the owner should be an international standard, a *de facto* international standard, or a national standard in use by the steel fabrication industry. To meet the requirements of a specific project application, the owner may use several standards. In addition, the contractual arrangements should identify clearly the responsibilities of all parties involved in the project. To this end, appropriate supplemental documents can be provided to correlate the special needs of the project with the relevant clauses of this International Standard and those of the selected standard(s).

Examples of standards that satisfy the requirements of 20.2.1 are

a) EEMUA 158[A.20.2-1] or other generic standards providing the guidance listed in 20.1,

b) AWS D1.1[A.20.2-2], generally compatible with the MC method and Annex E,

c) NORSOK M-101[A.20.2-3], generally compatible with the DC method and Annex F, and

d) CSA W59[A.20.2-4].

When welding procedure qualification by test is required (i.e. when the procedure is not pre-qualified, when comparable impact performance has not been previously demonstrated, or when the welding consumables are to be employed outside the range of essential variables covered by prior testing), qualification shall include Charpy V-notch testing of the as-deposited weld metal, impact tested in accordance with the selected standard.

A.20.2.2 Weld metal and HAZ properties

A.20.2.2.1 General

In addition to weld metal toughness, consideration should be given to controlling the properties of the heat affected zone (HAZ). Although the heat cycle of welding sometimes improves base metals of low toughness, this region will more often have degraded properties. A number of early failures in welded tubular joints involved fractures which either initiated in, or propagated through, the HAZ, often before significant fatigue loading.

The average HAZ toughness values in Annexes E and F have been found by experience to be reasonably attainable. Due to the large scatter usually observed, single specimen (one out of three) energy values of 7 J (5 ft-lbs) or lower are allowed without requiring retest. As criticality of the component performance increases, lower testing temperatures are usually required. More extensive sampling increases the likelihood of finding local brittle zones with low toughness values.

Since HAZ toughness is as much dependent on the steel as on the welding parameters, a preferable alternative for addressing this issue is through weldability pre-qualification of the steel. For example, API RP 2Z[A.20.2-5] spells out such a pre-qualification procedure, using CTOD as well as Charpy testing. This pre-qualification testing is presently being applied as a supplementary requirement for high-performance steels such as API RP 2W[A.20.2-6] and API RP 2Y[A.20.2-7] and is accepted as a requirement by several steel producers.

Conservative requirements in MC1 and DC structures and in API RP 2Z[A.20.2-5] on the subject of local brittle zones (LBZ) give little credit for ways and means of having managed to avoid LBZ induced failures in the recent past. These design countermeasures include

— the use of steels with moderate crack-arrest capabilities, as demonstrated by no-break in the NRL drop-weight test[A.20.2-8] (small flaw),

— overmatch and strain hardening in conventional normalized 290 MPa to 345 MPa (42 ksi to 50 ksi) carbon-manganese steels, in which the weld metal and HAZ are harder than adjacent base metal, forcing plastic strains to go elsewhere,

— the tendency for fatigue cracks in welded tubular joints to grow out of the HAZ before they reach appreciable size (assuming unfavourable osculation of joint can weld seam with the brace footprint is avoided, see Figure 20.2-1), and

— traditional limits on layer thickness or heat input in welding procedures, which promote grain refinement in the HAZ and avoid extensive LBZ.

NOTE Many standards permit testing one 345 MPa (50 ksi) steel to qualify all other grades of 345 MPa (50 ksi) and below. Consequently, selection of a premium grade of 345 MPa steel [such as API Spec 2H, 50 Z with very low sulphur, or 270 J (200 ft-lb) upper shelf Charpy] [A.20.2-9], for qualification test plates will virtually assure the satisfying of an HAZ impact requirement of 34 J (25 ft-lbs), even when welded with high heat inputs and high interpass temperatures.

A.20.2.2.2 Material category (MC) toughness

No guidance is offered.

A.20.2.2.3 Design class (DC) toughness

No guidance is offered.

A.20.2.2.4 Charpy V-notch (CVN) toughness

Charpy impact testing is a method for qualitative assessment of material toughness. Although lacking the technical precision of crack tip opening displacement (CTOD) testing, the method has been, and continues to be, a reasonable measure of fracture safety, when employed with a definitive programme of non-destructive examination to eliminate weld area imperfections.

The requirements specified in 20.2.2.4 are based on practices that have generally provided satisfactory fracture experience in structures located in moderate temperature environments [e.g. 4 °C (40 °F) sea water and − 10 °C (14 °F) air exposure].

For other environments, impact testing temperatures should be reconsidered, based on local temperature exposures. Since most welding procedure requirements are concerned primarily with tensile strength and soundness (with minor emphasis on fracture toughness), it is appropriate to consider additional essential variables which have an influence on fracture toughness (e.g. combinations of specific brands of wire and flux).

A.20.2.2.5 CTOD toughness

For critical welded connections, the CTOD testing is technically more exact than Charpy testing. CTOD tests are run at realistic temperatures and strain rates, representing those of the engineering application, using specimens having the full prototype thickness. This yields quantitative information useful for engineering fracture mechanics analysis and defect assessment in which the required CTOD is related to anticipated stress levels (including residual stress) and flaw sizes. When CTOD data for the HAZ is not provided by the steel manufacturer, the contractor can use weld procedure specification tests to CTOD qualify the welds necessary for the specific project. This is a shortened form of the API RP 2Z[A.20.2-5] pre-qualification testing usually performed by the steel manufacturer. These sections cover the testing requirements for the HAZ zone only.

Previous CTOD testing data performed by the steel manufacturer in accordance with a recognized international standard, e.g. API RP 2Z[A.20.2-5] or EN 10225[A.20.2-10], is an acceptable alternative for the HAZ requirements, provided the contractors' WPS is within the range of heat inputs, interpass temperature, thickness and other essential variables qualified.

A.20.2.2.6 Hardness testing

No guidance is offered.

A.20.2.2.7 Other mechanical tests

No guidance is offered.

A.20.2.3 Tubular T- ,Y- and K-joints

A.20.2.3.1 General

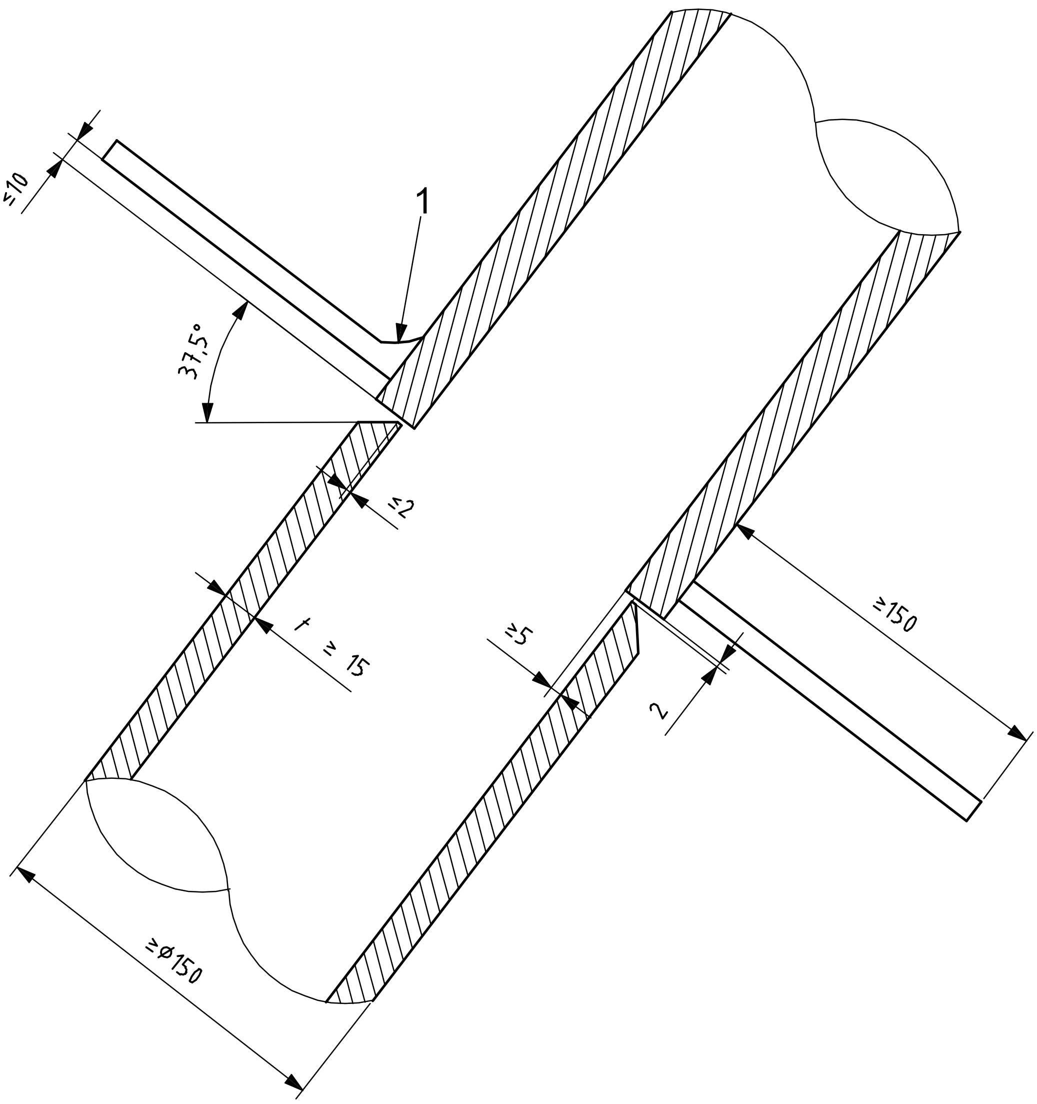
Specialized considerations for single-sided T-,Y- and K-joints are described comprehensively in AWS D1.1[A.20.2-2].

As examples, some of these considerations are briefly, though not comprehensively, discussed in A.20.2.3.2 and A.20.2.3.3.

A.20.2.3.2 Welder qualification

All welders making full penetration welds from one side only in tubular joints should qualify on a test which represents all welding positions and incorporates a shoulder at the open root, as well as restricted access from the chord side — for example, the 6GR test specimen as shown in Figure A.20.2-1. Typically, four prismatic bend tests are taken from the 45° locations. Macro-sections adjacent to the bend specimens should conform to the applicable visual and profile requirements.

Dimensions in millimetres



Key

1 disk tack weld

Figure A.20.2-1 — Welder test piece for single-sided welds

A.20.2.3.3 Production weld joint details

Examples of weld details for tubular T-,Y- and K- joints welded by shielded metal arc (manual stick) welding are shown in Figure A20.2-2 and presented in Tables A.20.2-1 and A20.2-2. Other processes are covered in References [A.20.2-1], [A.20.2-2] and [A.20.2-3].

Table A.20.2-1 — Root gap requirements for tubular joints - shield metal arc welding

Groove angle b	Root opening, g	
	mm	in
Over 90°	0 to 4,8	0 to 3/16
45° to 90°	1,6 to 4,8	1/16 to 3/16
Under 45°	3,2 to 6,4	1/8 to 1/4

Table A.20.2-2 — Minimum weld thicknesses for tubular joints - shield metal arc welding

Weld angle, a	**Weld thickness, t_w**
Over 135°	To continue line of brace/stub, except that t_w need not exceed 1,75 t Smooth profile required between brace/weld/chord
50° to 135°	1,25 t
35° to 50°	1,50 t
Under 35°	1,75 t

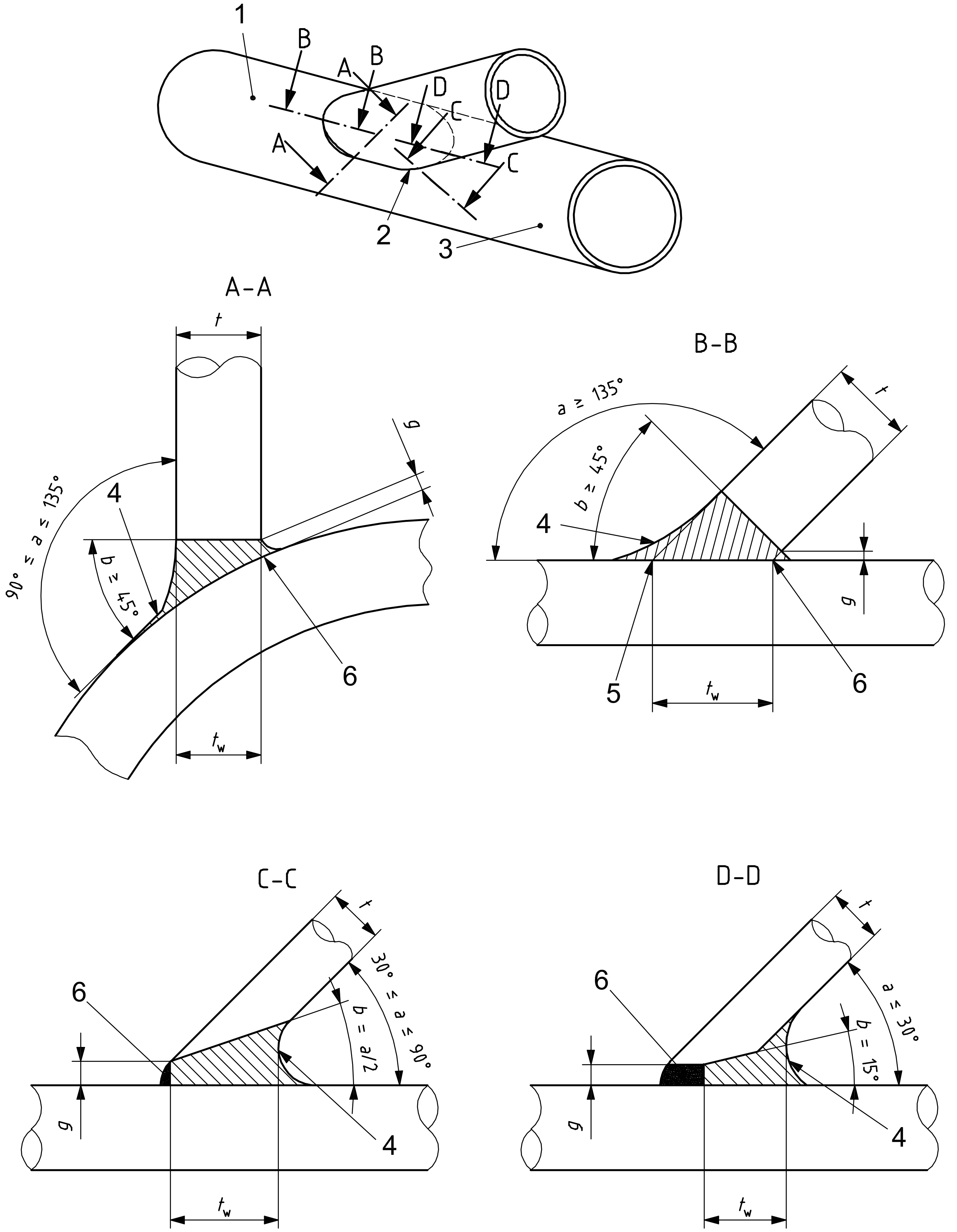


Figure A.20.2-2 — Welded tubular joints – shield metal arc (stick) welding

Key

1 chord
2 joint intersection and weld line
3 smooth transition between details C-C and D-D and external fillet of 1,75t
4 smooth profile between brace/weld/chord
5 build out to fit thickness, but t_w need not exceed 1,75t
6 weld root not subject to inspection beyond required t_w
a local dihedral angle formed by exterior surfaces of brace and chord at point being considered
b groove angle formed by brace weld preparation and chord at point being considered
g root gap, see Table A.20.2-1
t brace or stub thickness
t_w weld thickness, see Table A.20.2-2

Figure A.20.2-2 (*continued*)

A.20.3 Inspection

In some cases, it is not necessary to undertake 100 % inspection with all methods. The requirements given in Annexes E and F allow for inspection of a proportion of the length of the welds of certain types and for certain categories or classes of structure. The reduced extent of inspection is dependent on the welds being of good quality; consequently, if the defect identification is too high, the extent of inspection should be increased as below.

The defect rate is calculated from

$$\text{defect rate} = \frac{\text{sum of length of defects identified by method}}{\text{total weld length inspected by method}} \qquad \text{(A.20.3-1)}$$

The extent of NDT should be increased as follows:

— defect rate exceeds 5 % — proportion of welds inspected should be doubled, spot inspection proportion should increase to 20 %;

— defect rate exceeds 5 % for increased proportional inspection — proportion of welds inspected should increase to 100 %;

— defect rate exceeds 10 % — proportion of welds inspected should increase to 100 %.

The required level of increased extent should be maintained until a defect rate below 5 % is re-established and documented.

A.20.4 Fabrication

No guidance is offered.

A.21 Quality assurance, quality control and documentation

A.21.1 General

Exposure level L1 structures have the most severe failure consequences and hence the most stringent inspection and documentation requirements.

In selecting the inspection category, welds in joints more than 150 m below the water surface should be assumed inaccessible for in-service inspection by divers. Accessibility for inspection at greater depths may be considered if the use of ROV methods (e.g. visual, wet MPI, or flooded member detection) is planned.

When only partial testing is required for welds in an area, the weld testing should be distributed such that the most essential members and nodes are included in the inspection and such that areas of welds most susceptible to weld defects are examined.

A.21.2 Quality management system

Conformance and/or accreditation to a formalized quality management system is preferred, e.g. ISO 9000 with ISO 9001 and ISO 9004, API Q1[A.21.2-1], or similar recognized standard.

The requirements of Table 21.2-1 should be considered even when formalized QMS documentation is not required.

A.21.3 Quality control plan

A.21.3.1 General

Proactive QC by the contractor is different from after-the-fact QA inspection, testing and documentation, which is normally conducted by the owner or by a 3rd party.

The use of “negative reporting” systems for QC inspection, whereby a formal report is only raised when a defect is found, may be considered, subject to the contractor furnishing evidence of appropriate procedures controlling such systems to the owner’s satisfaction. Available logs, markings or other records for both positive and negative QC inspection outcomes should be retained — at least temporarily, in any case.

Further considerations for the QC plan are presented in the following subclauses.

A.21.3.2 Inspector qualifications

Inspectors should be knowledgeable in the general areas of welding technology, inspection and testing procedures, as well as in construction methods for those areas of their responsibility during fabrication. They should know how and where to look for problems and situations which lead to problems, as well as the practical limitations on making repairs.

Suitable standards for the qualification of inspectors include

— AWS QC-1[A.21.3-1],

— CSA Standard W178.2[A.21.3-2],

— CSWIP[A.21.3-3],

— BGAS-CSWIP[A.21.3-4], and

— PCN[A.21.3-5].

Other standards may be agreed upon between owner, contractors and regulators.

A.21.3.3 NDT personnel qualifications

All NDT personnel should have demonstrated ability and experience, and should preferably be qualified to an internationally recognized offshore code or guidance. Continuing qualification should be based on satisfactory performance on the job.

Internationally recognized requirements for NDT personnel are contained in

— CSWIP[A.21.3-3],

— BGAS-CSWIP[A.21.3-4],

— PCN[A.21.3-5],

— ACCP (ASNT)[A.21.3-6],

— SNT-TC-1A (ASNT)[A.21.3-7], and

— ISO 9712[A.21.3-8].

Additionally, API RP 2X[A.21.3-9] provides specific offshore guidance.

Personnel who perform other types of inspection during the fabrication of an offshore structure should be required to demonstrate ability and experience, or be qualified to an appropriate code for the required inspections.

A.21.3.4 Inspection of materials

Receipt of the correct material should be verified by cross-checking with appropriate original mill certificates and heat stamps, or with other appropriate documentation for non-structural material and structural materials other than steel.

A.21.3.5 Inspection of fabrication

In general, the inspection of fabrication should confirm that each component incorporated into the structure is of the correct material, orientation, size and dimension, etc.; and is fitted, aligned, and permanently fastened according to the specified requirements. Legs and pile sleeves through which piles will be field installed should be carefully checked for internal clearance and straightness, to ensure required tolerances have been met and, if possible, a template of the pile of nominal length should be passed through the leg or sleeve to demonstrate adequate clearance. Particular attention shall be given to field joints (such as the tops of structure legs) which should be checked to ensure all dimensions are within tolerance.

A.21.3.6 Inspection of welding

Inspection and testing of welding should be performed during all phases of fabrication, with an aim to preventing introduction of defects into the weld. To the maximum extent possible, inspection and testing should be performed as construction progresses and be scheduled so as not to delay the progress of the job. All weld fit-ups (joint preparation prior to welding) should be visually inspected to ensure acceptable tolerances before welding.

It should be verified that the welder or welding operator is currently qualified for the procedure being used and that the appropriate qualified procedure is being followed. In addition, inspection should ensure that appropriate consumables are being used and that the consumables are being stored, handled and used in accordance with the manufacturer's recommendations and other appropriate requirements.

A.21.4 Inspection of installation aids and appurtenances

No guidance is offered.

A.21.5 Inspection of loadout, sea-fastening and transportation

No guidance is offered.

A.21.6 Installation inspection

No guidance is offered.

A.21.7 Documentation

A.21.7.1 General

In addition to document preservation, document control should have an information flow from contractors to owners, and vice versa, and be able to track and distribute revisions and drawing issuances in a timely manner.

Document retention for the life of the structure is generally the responsibility of the owner. Permanent records should not be put into a “destroy after X years” system.

The following considerations apply only where indicated in Table 21.7-1.

a) **Weight and centre of gravity reports**

The contractor should prepare a weight and centre of gravity report for each structure. Weights should be based on design, shop or as-built drawings as the job progresses. Guidance on weight control is given in ISO 19901-5[5].

b) **Dimensional control documentation**

Prior to and during fabrication, documentation should be prepared on the dimensions and compliance of components and of assemblies.

c) **Materials and fabrication inspection**

During the fabrication phase, material inspection documentation covering the mill certificates and material identification records (as described in Clause 19), as well as any additional materials, testing or special inspections which were conducted and which reflect on the final condition of the structure, should be prepared and compiled. This need not include documentation for inspection related to the assembly of the structure.

d) **Weld inspection**

A set of structural drawings should be marked with an appropriate identification system detailing the location of each weld to be examined and referenced as an integral part of the inspection record. All welds should be uniquely identified and be traceable during fabrication to the individual welder or welding operator. A report should be prepared for each non-destructive test performed (other than visual), the details of which shall be documented sufficiently to permit repetition of the examination at a later date. Sketches and drawings incorporating the weld identification system should be used to augment descriptions of the part and locations of all discontinuities required to be reported. Forms should be provided to show the required details of documentation, and sketches of typical weld configurations should also be provided to clarify the written description. Discontinuities required to be reported should be identified on sketches by the appropriate weld number and position.

e) **Other inspection**

Inspection of all non-structural systems and tests should be documented to confirm details of the inspection and results. Any deviations from the specified requirements shall be properly recorded, including sketches if necessary.

A.21.7.2 Calculations

No guidance is offered.

A.21.7.3 Weight and centre of gravity reports

No guidance is offered.

A.21.7.4 Fabrication inspection documentation

No guidance is offered.

A.21.7.5 Installation inspection documentation

No guidance is offered.

A.21.8 Drawings and specifications

A.21.8.1 General

Drawings should be prepared using consistent units.

For use in connection with fixed offshore structures and related facilities, the drawings and specifications are defined as described in the following subclauses.

A.21.8.2 Conceptual drawings

Conceptual drawings are intended to supply a general idea of the facility under consideration. These drawings should include preliminary layouts and elevation views of the overall facility showing the number, type of construction and approximate size of each platform, as well as the more important auxiliary features, such as heliports and boat landings.

Simplified process or mechanical flow diagrams and electrical one-line diagrams should be included for all production or utility systems. A generalized equipment layout drawing should be included which also indicates buildings, storage of supplies, etc.

All information which contributes to clarifying the overall intent of the facility should be shown. Specifications are not generally required. However, if included, they should be of a general descriptive nature, so as to supplement the drawings in adequately describing the facility.

A.21.8.3 Bid drawings and specifications

Bid drawings are intended to show the total facility with its configuration and dimensions in sufficient detail to accurately define the scope of the project. With supplemental specifications, bid drawings are suitable for generally defining the scope of the proposal, or suitable to be furnished by the owner to a contractor when requesting a quotation in which the design is to be part of the contractor's bid. In the latter case, all essential information needed by the designer should be included.

Bid structural drawings should show major overall dimensions, deck arrangements, operational loading requirements and any preferred type of construction and materials. Structural details and member sizes are not necessarily furnished, since these are considered “design” drawings. All auxiliary items which are to be included in the bid, such as boat landings, barge bumpers, stairs, walkways, fence and handrail, should be shown on these drawings. Typical preferred construction details of these items also should be included.

Equipment layout drawings should be included for all decks. Sufficiently detailed process, mechanical and utility flow diagrams and electrical one-line diagrams should be included for all systems which are covered by the bid.

Specifications for equipment, machinery and other engineered components should include an itemized list and description of all items not shown in the drawings, but which are to be included in the bid — even such items as lighting and cathodic protection. Specifications for materials and fabrication should include all types of material allowed for use and any particular requirements for dimensional tolerances, inspection, testing and welding.

A.21.8.4 Design drawings and specifications

Design drawings give descriptive information about the major components of the facility. Emphasis in these drawings is placed on overall layouts and definition of critical items, supplemented by essential details. They should indicate all appurtenances and should include all dimensions where strict adherence is required.

Design drawings should include a layout of the location and orientation of the structure or structures in the field, as well as the location of equipment on the decks of each structure. Structural drawings showing member sizes of all major structural members and all controlling dimensions should be included. General locations and preliminary or typical details of miscellaneous structural items, such as joints, cover plates and web plate stiffeners, should be indicated. In addition, any other typical structural details which are not normally standard to this type of construction should be included.

Design drawings should also include all items necessary for installation purposes, such as lifting eyes and launch frames, which are critical to the design of the structure.

Mechanical and utility flow diagrams showing sizes of all equipment, piping and valves, and electrical line diagrams showing rating and sizes of feeders and controls, should be included. Equipment layout drawings of all equipment shown on the flow diagrams or one-line diagrams, manifolds and major instrumentation items, such as large control valves, meter runs, control valve stations and control panels, should be shown. Piping plan and elevation drawings should show major piping only and indicate adequate space reserved for minor piping and for conduit and cable runs.

Design drawings should be supplemented by all specifications necessary to convey the intent of the design. Standard specifications for material and fabrication which are referred to in this International Standard can be properly referenced on appropriate drawings. However, any deviations from these specifications should be detailed. Specifications should be included for equipment, machinery and other engineered items.

Design drawings and specifications are often used as part of the solicitation package, or as part of the contract document. As such, they need to be sufficiently detailed and suitable for furnishing by the owner to the contractor to be used for making accurate material take-offs for bidding purposes when no design is required on the part of the contractor, or suitable for submittal by the contractor to the owner to completely define the proposal. When design drawings are used for bid or contract purposes, all auxiliary items such as stairs, boat landings and walkways should be shown in sufficient detail for estimating purposes.

A.21.8.5 Fabrication drawings and specifications

Fabrication drawings are intended to supply sufficient information that fabrication can be performed directly from these drawings. They should contain all design data fully detailed and dimensioned. At the fabricator's option, they may be supplemented by shop drawings.

A set of fabrication drawings includes completely detailed design drawings with descriptions, exact locations, sizes, thicknesses and dimensions of all structural members and stiffeners. This information should also be shown for all structural items, such as brackets, stiffeners and cover plates, and for all auxiliary items, such as stairs, walkways, fence and handrail. Connections and joints should be completely detailed, including welding symbols, unless standard procedures apply. Methods of attaching timber, grating and plate should be included.

In addition to complete piping plan and elevation drawings, a set of fabrication drawings should include piping isometric drawings and details for all pipe supports, if required by the complexity of the facility. Instrumentation location plans and supports, electrical location diagrams showing general routing, and wire and cable tie-ins to electrical equipment should be included.

Fabrication drawings should clearly indicate the components or “packages” scheduled for assembly as units in the fabrication yard. Welds and connections to be performed in the field should be indicated.

Detailed specifications should be included for all work to be done by the fabricator, such as welding, fabrication and testing, and for all materials, equipment or machinery to be purchased by the fabricator. However, for standard specifications covered under the recommendations of this International Standard, no copies need be furnished, provided reference is made on key drawings. Specifications for equipment and other engineered items not purchased by the fabricator may also be included with fabrication drawings, for general information.

A.21.8.6 Shop drawings

Shop drawings or sketches are prepared by or for the fabricator, at his option, to facilitate the fabrication of parts and/or components of the structure. They are intended to provide all information and instructions needed for that purpose. Due to differences in methods and procedures of various fabricators, shop drawings can vary in appearance.

Shop drawings may include typical shop details to supplement details and dimensions shown on either fabrication drawings or patterns for coping the ends of members, detailed piece marked drawings for each member and pipe spool drawings.

Shop drawings are the responsibility of the fabricator. Approval or review of shop drawings by the designer or owner should not relieve the fabricator of his responsibility to complete the work in accordance with the contract or fabrication drawings and specifications.

A.21.8.7 Installation drawings and specifications

Installation drawings furnish all pertinent information necessary for the construction of the total facility on location at sea. They contain relevant information not included on fabrication drawings.

If special procedures are required, a set of installation drawings may include installation sequence drawings. Details of all installation aids such as lifting eyes, launch rails or frames, jacking brackets and stabbing points should be included, if not shown on fabrication drawings. For structures installed by flotation or launching, drawings showing launching, upending and flotation procedures should be provided. Details should also be provided for piping, valving and controls of the flotation system, closure plates, etc.

Erection of temporary struts or supports should be indicated. All rigging, cables, hoses, etc. that are to be installed prior to loadout should be detailed. Barge arrangement, loadout and tie-down details should be provided.

Installation drawings are intended to be used in connection with fabrication drawings. They should be supplemented by detailed installation specifications, installation procedures or special instructions, as required, to provide all information required to complete the field installation.

A.21.8.8 As-built drawings and specifications

As-built drawings show in detail the manner in which the structure was actually constructed. These drawings are usually made by revising the original fabrication drawings, supplemented by additional drawings if necessary. As-built drawings are intended to reflect all changes, additions, corrections or revisions made during the course of construction. They are prepared for use by the owner to provide information related to the operation, servicing, maintenance and future expansion of the facility.

When the preparation of as-built drawings has been authorized by the owner, it is the responsibility of the fabricator and the field erector to furnish to the owner or to the designer adequate information regarding all variations between the drawings and the facility as actually constructed. This is usually furnished as corrections from the yard, the shop and the field, marked on prints of the original drawings or by supplementary sketches, if required. This information should be sufficiently complete that the owner or the designer can correct and revise the original drawings without additional data or field measurements. Since the fabricator and erector are responsible for the accuracy of the corrections, a review and/or approval of the corrected drawings should be made by both the fabricator and the erector.

Minor deviations from the original drawings are generally numerous. Differences between the actual dimensions and those shown on the drawings need not be reported if they are within the specified allowable tolerances.

Specifications should also be corrected to reflect any changes made during the purchase of material, equipment, or machinery.

A.22 Loadout, transportation and installation

A.22.1 General

No guidance is offered.

A.22.2 Loadout and transportation

No guidance is offered.

A.22.3 Transfer of the structure from the transport barge into the water

No guidance is offered.

A.22.4 Placement on the sea floor and assembly of the structure

No guidance is offered.

A.22.5 Pile installation

A.22.5.1 General

No guidance is offered.

A.22.5.2 Stabbing guides

No guidance is offered.

A.22.5.3 Lifting methods

No guidance is offered.

A.22.5.4 Field welds

No guidance is offered.

A.22.5.5 Driveability studies

References [A.22.5-1] to [A.22.5-18] provide relevant information on driveability analyses and the parameters used in these analyses.

A.22.5.6 Obtaining required pile penetration

No guidance is offered.

A.22.5.7 Driven pile refusal

Two examples of driven refusal criteria are given below.

a) In soft soils, pile driving refusal for a properly operating hammer is defined as the point where pile driving resistance exceeds either 1 000 blows/m (300 blows/ft) for a consecutive 1,50 m (5 ft) of penetration, or 800 blows for 300 mm (1 ft) of penetration. This definition applies when the weight of the pile does not exceed four times the weight of the hammer ram. If the pile weight exceeds this, the above blow counts are increased proportionally, but in no case should they exceed 800 blows for 150 mm (6 in) of penetration.

b) In hard clays and dense sands, pile driving refusal can be defined as the point where driving resistance exceeds one of the following criteria:

- in continuous driving — a minimum of 125 blows/250 mm over six consecutive intervals of 250 mm, or a minimum of 200 blows/250 mm over two consecutive intervals of 250 mm;
- in the last interval of 250 mm at the end of driving — 325 blows/250 mm;
- at restart of driving after a stoppage for one hour or longer — 325 blows/250 mm over two consecutive intervals of 250 mm.

In soils where hard driving conditions are anticipated, such as in the presence of boulders or of strong cemented layers, the definition of pile refusal criteria cannot be based solely on a blow count value, and the potentially high local driving stresses induced in the pile should also be taken into account. The stress level in the pile steel can be calculated from wave equation analyses, and can be estimated from the stress measurements from pile instrumentation. An example of refusal criteria for pile driving in strongly cemented carbonate soils is given in References [A.22.5-19] and [A.22.5-20].

The potential consequences of hard driving conditions in strong cemented layers (i.e. damage of the pile, hammer or structure) are highly dependent on the hammer type and size, on the pile wall thickness (D/t ratio, presence of a driving shoe), and on possible defects and irregularities in the pile shape, as well as on the soil conditions (in particular, strength and thickness of the rock layer, and soil type below the rock formation). Moreover, the reflected stress level (ratio of the maximum reflected stress to the initial peak stress), as measured from pile instrumentation at the pile head, only gives an estimate of the average stress in the pile wall; more severe stresses can be experienced locally at the pile tip during driving. Therefore, the definition of driven pile refusal criteria in cemented soils should preferably be based on local piling experience at the site. Correlation charts, similar to the one proposed in Reference [A.22.5-12], can be developed as an aid in deciding whether pile driving through a cemented layer can be attempted, or if drilling of the rock below the pile tip is necessary.

A.22.5.8 Pile refusal remedial measures

No guidance is offered.

A.22.5.9 Selection of pile hammer

No guidance is offered.

A.22.5.10 Drilled and grouted piles

No guidance is offered.

A.22.5.11 Belled piles

No guidance is offered.

A.22.5.12 Grouting pile-to-sleeve connections and grouted repairs

No guidance is offered.

A.22.5.13 Pile installation records

No guidance is offered.

A.22.5.14 Use of hydraulic hammers

No guidance is offered.

A.22.6 Installation of conductors

No guidance is offered.

A.22.7 Topsides installation

No guidance is offered.

A.22.8 Grounding of installation welding equipment

No guidance is offered.

A.23 In-service inspection and structural integrity management

A.23.1 General

A.23.1.1 Applicability

Clause 23 provides general guidance for inspection of fixed steel offshore structures

- located anywhere in the world,
- built to any design code,
- analysed to any degree of engineering sophistication,
- fabricated with any welding procedure/specification,
- installed in any year,
- subject to any operational history,
- possibly containing design deficiencies or fabrication defects, and
- possibly experiencing degradation or damage from a variety of sources.

The primary objectives of this International Standard are to safeguard human life and to protect the environment. The default inspection requirements of 23.7 are based on consideration of these two objectives. However, the owner also has the responsibility and prerogative to consider economic impact and property interests in deciding whether additional inspection is needed to achieve a desired level of structural reliability and risk management objectives. Such economic interests can include prevention of lost hydrocarbon production resulting from platform shutdown.

National standards and/or statutory requirements can specify stricter inspection requirements, reflecting special national or regional interests or priorities for protecting human life and the environment, minimizing waste of natural resources (such as oil and gas), preventing general economic disruption, etc. Annex H contains certain regional information.

A.23.1.2 Inspection motives

Table A.23.1-1 provides examples of basic inspection motives (root causes of the need for inspection).

Table A.23.1-1 — Examples of inspection motives

Motive:	Examples
Fabrication defects or installation damage	Weld defects (major and minor), material imperfections, dents, deformations
Degradation or deterioration	Corrosion, fatigue, scour, subsidence, sea floor instability
Design uncertainties or errors	Approximations (especially for oceanographic and seismic data), analysis uncertainties (especially for fatigue), underdesign, marine growth build-up
Environmental overload	Storm, earthquake, mud slide, tsunami, ice
Accidental events	Vessel impact, dropped objects, explosion, abrasion, floating debris, riser damage from anchor drag on pipeline
Modifications from original purpose	Addition of personnel, topsides equipment, support framing, risers or conductors, structure life extension
Repairs	Clamps, wet welds, bolting, adhesives

A.23.1.3 Inspection in structural integrity management

In-service inspection is an integral part of overall structural integrity management which, as illustrated in Figure A.23.1-1, is an ongoing process for ensuring the structural integrity and fitness-for-purpose of an offshore structure or group of structures. The four phases of structural integrity management (SIM) are defined by Figure 23.1-1. The terms *evaluation*, *inspection strategy*, and *inspection programme* are used in this clause with the specific meanings indicated in the figure.

A key feature of SIM is the maintenance, transfer and use of data obtained from the various phases of SIM. Informed decisions and management of change are best made through the use of such data, thereby ensuring that the desired performance levels can be reached.

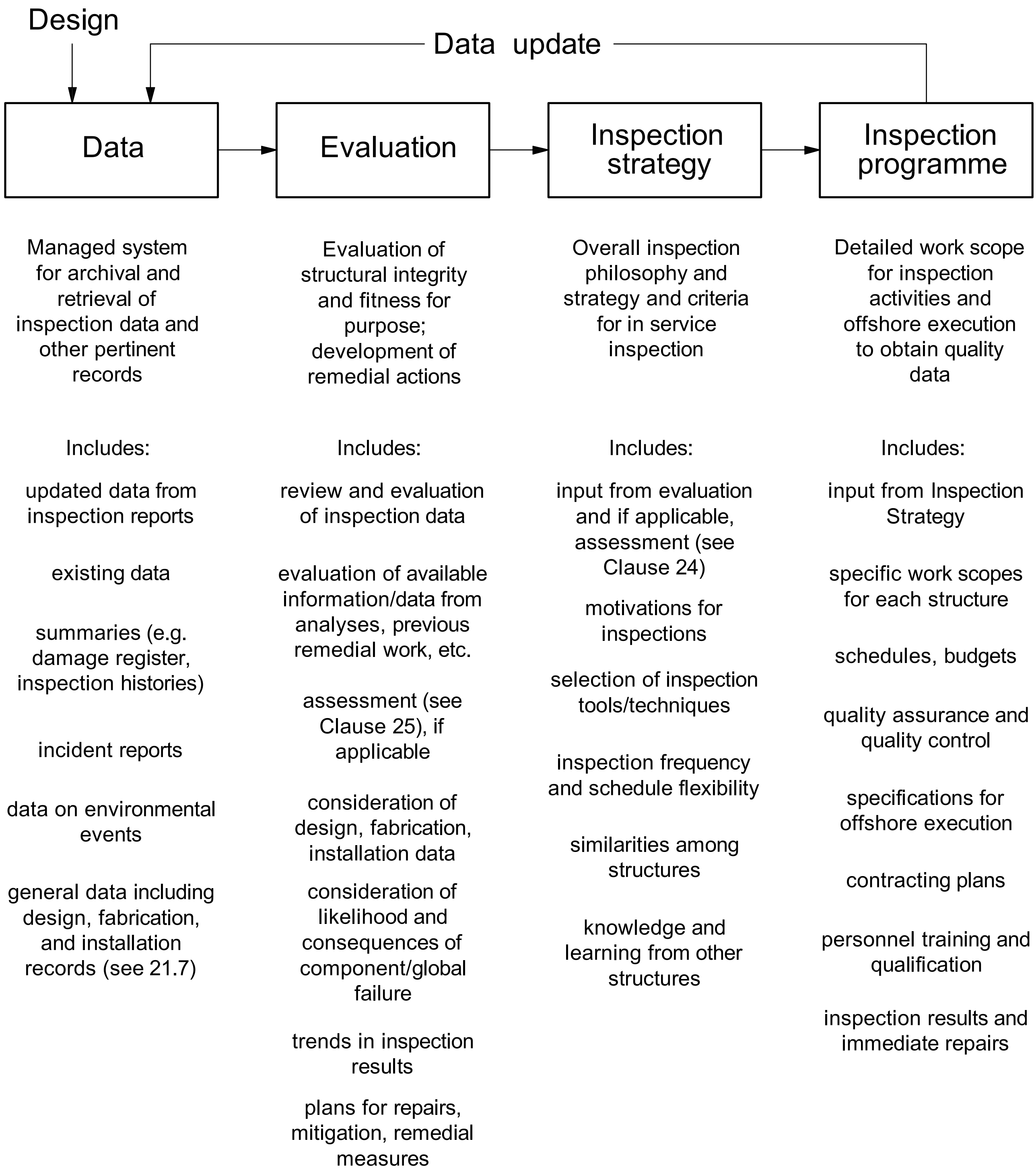


Figure A.23.1-1 — Structural integrity management

Potential benefits of SIM include the following.

— Prioritization of inspection resources.

Structures and components are prioritized on a strength, risk or reliability basis. Consideration should be given to structure or component criticality to strength, fatigue, likelihood of failure and consequence of failure.

— Increased knowledge of assets.

  SIM requires review of all available data and assessments by qualified personnel. Hence knowledge of the structure's condition, strength and fatigue resistance is gained.

— More effective management of change.

  All records are reviewed and maintained, thereby allowing transfer of knowledge and learning for the owner and improving decisions.

— Planned maintenance in lieu of on-the-spot offshore repairs or modifications.

— Increased knowledge of a structure's condition, strength and fatigue resistance can allow sufficient time to properly engineer repair. Review of assessment can result in delayed or no repair.

A.23.2 Data collection and update

The importance of maintaining structure and inspection data cannot be over emphasized. Evaluations (see 23.3) and assessments (see Clause 24) are only as accurate as the engineering methodology and the data used therein. Missing or incorrectly measured data can force the making of conservative assumptions during an engineering assessment. Examples of insufficient data affecting the potential or perceived integrity of a structure include

— lack of knowledge of the structure that can prevent the addition of additional facilities if spare capacity cannot be exploited, and

— lack of information on a dent depth and location that requires the assumption that the dent is located where it will cause the highest strength reduction.

Maintaining good quality data in a retrievable manner can therefore permit modifications to a structure or avoid the necessity of more detailed assessment. To this end, owners should develop specifications which detail underwater measurement techniques, personnel qualifications, survey limits, anomaly criteria, etc.

To facilitate periodic evaluation and consequent updates of the inspection strategy, the following record keeping practices are recommended.

— The maintaining of an up-to-date listing of basic information about each structure in the owner's inventory.

  EXAMPLE Name, date installed, water depth, location, basic function and configuration, pipeline connectivity, design basis, design premise, references for design/fabrication/installation information, listing of modifications/additions/repairs.

— The maintaining of an up-to-date listing of basic information about each inspection performed on each structure

  EXAMPLE Structure name, date inspection was performed, basic scope of work, important results, available reports, documentation.

— The maintaining of an up-to-date record of inspection trends for each structure

  EXAMPLE Variation in cathodic potential readings over time, monitoring for change in condition of known damage/defect.

— The maintaining of an up-to-date summary of reported anomalies

  EXAMPLE Damage, defects.

— The entering of new information promptly into the record keeping system.

Records of all in-service inspections should be retained for the life of the structure and should contain complete detailed information about

— the inspection tools and techniques employed,

— the actual scope of work (including any field changes) for each of the tools and techniques employed,

— the inspection data collected including photographs, measurements and videotapes,

— the inspection findings, including thorough descriptions and documentation of any anomalies discovered, and

— the inspection level completed.

Any resulting repairs and engineering evaluations of the structure's condition should be documented and retained by the owner.

Fabrication and installation inspection data also provide important information about the initial structure condition and have a direct bearing on the in-service inspection strategy. Record keeping requirements for these inspections are given in 21.6.

All information listed above should be transferred to any new owner as necessary.

A.23.3 Evaluation

A.23.3.1 General

The following points clarify the distinction between evaluation as required by Clause 23 and assessment as required by Clause 24.

a) Evaluation is an ongoing process, whereas assessment is triggered by certain initiators. Evaluation does not necessarily imply computer structural analysis. Evaluation can include, but is not limited to, complex analysis, simplified analysis, engineering judgment, experience and use of experimental results.

b) Upon receipt of inspection data, an evaluation can be carried out, even if there are no defects found. For example, it can be appropriate to modify the inspection strategy in light of trends in cathodic potential readings.

c) When damage, defects or deterioration are found, an evaluation is required to determine, for example, whether a primary component is affected (in which case an assessment can be needed), or if more inspection is needed on an urgent basis to further define the extent of the damage.

Some similarities between the two are

— much of the data required for evaluation is the same as that for assessment (analysis results, environmental data, etc.), and

— evaluation can consider possible ways (e.g. remedial measures given in A.23.3.5) to eliminate a problem or reduce it to a manageable level without having to conduct an assessment. The assessment process specified in Clause 24 also allows for the use of remedial measures in lieu of the pursuit of further assessment stages.

Examples of evaluation methods can be found in Reference [A.23.3-1]

A.23.3.2 Risk assessment

The evaluation, consequent inspection strategy and any remedial measures should meet the objectives of the structural integrity management system in maintaining fitness-for-purpose. The risk matrices and structure exposure levels given in 6.6 provide a useful basis but should be supplemented with additional assessment of the risk if the overall structure rating is too coarse or general to address specific concerns, aspects of performance or individual components. In all cases, consideration should be given to the following points:

a) the consequences of structure or component failure, including consideration of

   1) platform manning,

   2) well characteristics (naturally flowing, sour gas, high pressure, etc.),

   3) pipeline characteristics (content, safety valves, etc.),

   4) proximity of platform to environmentally sensitive areas, and

   5) criticality of the platform to other operations;

b) the perceived likelihood of such failure, including consideration of

   1) characteristics of environmental actions,

   2) vulnerability to accidental loading (e.g. proximity to shipping lanes),

   3) present structural condition,

   4) degradation mechanisms,

   5) service history,

   6) reserve strength,

   7) structural redundancy and alternative load paths, and

   8) fatigue sensitivity.

A.23.3.3 Structural considerations in evaluation

Table A.23.3-1 lists some of the factors to be considered in evaluating the structural strength and fatigue performance of fixed steel offshore structures for the purposes of conducting an evaluation or developing an inspection strategy.

Table A.23.3-1 — Evaluation considerations for inspection strategy

Reference in 23.3.3	Considerations	Factors
a)	Structure age, condition, original design situations and criteria and comparison with current design situations and criteria	Remaining service life, desire to extend service life Platform operating and maintenance personnel should be consulted to see if they have observed conditions (corrosion evidence, movement in conductor guides or riser/J-tube/caisson supports, excessive deformations or deflections, unusual vibrations, change in platform sway response to waves, etc.) that should be evaluated
b)	Analysis results and assumptions for original design or subsequent assessments	Computed utilizations and fatigue lives Original design code and version Degree of sophistication and conservatism in the design analyses Amount of conservatism in design implementation, acceptance criteria Intentional overdesign for fatigue to reduce periodic inspection requirements Material specification
c)	Structure reserve strength and structural redundancy	
d)	Fatigue sensitivity	
e)	Degree of conservatism or uncertainty in specified environmental conditions	Data source Degree of certainty or conservatism in environmental conditions (wave, current, wind) and design assumptions (marine growth, earthquake spectra) Sensitivity of storm actions to return period. For example, how much difference in magnitude of actions is there between the 10 year, 100 year, and 1 000 year events? Relative severity of sea states for fatigue and storm conditions, since fatigue tends to be important where operational sea states are not far below design storm conditions Marine growth type (hard, soft), percent coverage, thickness, variation with depth, roughness
f)	Extent of inspection during fabrication and after transportation and installation	
g)	Fabrication quality and occurrences of any rework or rewelding	Unusual or special circumstances, rework/rewelding, wind induced vibrations/fatigue Extent of inspection during fabrication Fabrication quality Welding procedures and specifications
h)	Damage (including fatigue damage) during transportation or installation	Occurrence of any damage or vibrations during transportation Extent of inspection after transportation Severity of transport conditions and actual exposure (for example transoceanic versus local tow) Occurrence of any damage during installation Extent of inspection after installation Extent of deviations from design assumptions (e.g. air gap between deck and mean sea level)

Table A.23.3-1 (*continued*)

Reference in 23.3.3	Considerations	Factors
i)	Operational experience, including previous in-service inspection results and lessons from performance of other structures	Degree of vigilance in reporting/evaluating accidental events. Extent of deviations from design assumptions (e.g. sea states, marine growth, platform purpose) Modifications and additions of risers, service caissons, topsides, etc. Occurrence of any damage Absolute years of service Years of service relative to design service life Subsidence Scope of prior inspections Tools and techniques used Anomalies discovered Trends identified Failures or problems encountered with certain components under certain conditions Success of similar structures in same locale/region
j)	Modifications, additions and repairs or strengthening	Underlying causes necessitating repair or strengthening In-service performance of repairs or strengthening
k)	Occurrence of accidental and severe environmental events	
l)	Criticality of structure to other operations	
m)	Structure location (geographical area, water depth)	Particular regional experience
n)	Debris	
o)	Structural monitoring data, if available	
p)	Potential reuse or removal intents	

For older structures, the age of the design provides a clue to possible deficiencies or conservatisms in the design that should be considered in an inspection programme. As offshore technology has evolved, lessons learned from problems in the field, as well as new experimental data and analytical capabilities, have been incorporated in updated versions of various design codes. For example, significant improvements have been made in areas, such as

— design requirements for joint cans,

— material selection,

— wave force calculation procedures, and

— S–N curves.

Cracking is not generally experienced, if cracks occur, they are most likely found at

— joints in the first horizontal conductor framing below the water surface (normally resulting from fatigue),

— the main brace to leg joints in the vertical framing at the first bay above the sea floor (normally due to environmental overload),

— the perimeter members in the vertical framing at the first level below water (normally as a result of boat impact), or

— poorly designed connections in which the arrangement of components, accessibility and quantity of weld metal required make good weld quality difficult to achieve.

The extent of knowledge about environmental conditions varies for different regions of the world. The degree of certainty in establishing design data for sea states, current, wind, marine growth, seismicity and corrosion rates is not uniform. Thus, the amount of conservatism needed or desired for such data can vary for different regions, even among different owners within a given region. Inspection planners should be cognizant of these uncertainties and conservatisms.

Recently, some North Sea owners have adopted a strategy of intentionally overdesigning components of a structure for fatigue (such that the computed fatigue lives are 10 times higher than the design service life) with the goal of reducing/eliminating the need for Level III inspection. Such strategies are legitimate ways of trying to minimize the life-cycle cost of inspection; however, level III inspections are recommended for new structural concepts — at least until sufficient experience of the performance has been gained. The strategy of over designing for fatigue does not preclude the need for Level I and II inspections.

The quality of structure fabrication and extent of inspection during fabrication and installation has a direct bearing on the strategy for in-service inspection. One of the main motives for in-service inspection is detection of unknown fabrication defects (usually in weldments) or installation damage. If the incidence of such defects could be reduced through increased fabrication/installation inspection, better quality materials and improved welding procedures, in-service inspection requirements would be decreased. Specific guidelines are provided in Clause 21 for fabrication/installation inspection and in Clauses 19, 20 and 21 for fabrication quality. Owners have the flexibility to adopt stricter practices in these areas so as to reduce in-service inspection requirements.

Inspections can be more efficient and cost-effective when planned with detailed knowledge of the operational history and design/fabrication peculiarities of the structure.

In-service inspection requirements can also be affected (both positively and negatively) by lessons learned from the performance of other structures. Such knowledge provides an incentive for owners to cooperate and share technical lessons from inspections to benefit the industry as a whole.

A.23.3.4 Corrosion control considerations in evaluation

Listed below are some of the factors to consider in evaluating the corrosion protection performance of fixed steel offshore structures for the purposes of conducting an evaluation or developing an inspection strategy.

a) Environmental conditions:

   — water temperature as a function of water depth;

   — extent of marine growth and calcareous scale;

   — depolarization from major storms.

b) Design and analysis (sacrificial anodes):

   — anode size/proportion, material composition, and distribution/placement;

   — accuracy in estimate of steel surface area;

   — existence of complicated geometries, shadowing effects;

   — philosophy for protection of pipeline risers, conductors, large mudmats, and other structure components that can require separate or supplemental protection;

- amount of conservatism in design implementation, acceptance criteria;
- grade of steel and vulnerability to hydrogen embrittlement from overprotection.

c) Operational history:

- modifications and additions affecting steel surface area underwater;
- presence of significant debris acting as drain on anode performance;
- occurrence of any damage (loss of anodes due to pile driving vibrations or during installation);
- years of service.

d) Previous inspection findings:

- trends in cathodic potential readings over time;
- anode depletion estimates;
- local areas of relative high or low cathodic potential readings;
- type of probe used (proximity versus contact) and reliability of calibration;
- possible interference from dissimilar metals on ROV.

A.23.3.5 Remedial measures

Should a defect be found or if a design is perceived as unacceptable, remedial measures that reduce the risk of structural failure can be implemented either in isolation or in conjunction with each other. The choice of measures and their extent will depend on the source of risks to structural integrity and their magnitude. Remedial measures which can be considered include:

- change to operational procedures (e.g. supply boat movements);
- de-manning criteria;
- inspection of other components, or similar structures;
- more detailed or frequent inspection of defects or damage;
- remedial grinding of crack-like indications;
- repair of identified damage or defects;
- load reductions (e.g. marine growth removal);
- strengthening;
- structure assessment in accordance with Clause 24.

A.23.4 Inspection strategy

A.23.4.1 Basis

Development of an inspection strategy provides a basis for flexibility in extent and scheduling of periodic inspection programmes for a given structure. Standardization of work scopes, tools and techniques, and execution procedures can promote consistent quality and reporting and lead to efficiencies.

Two contrasting approaches to periodic inspection are described below.

a) A significant commitment to ongoing in-service inspection, with the goal of reducing the possibility of major repairs (clamps, member replacements) in the future. This approach relies on early detection of damage and defects with prompt implementation of relatively inexpensive repairs and preventive measures (grinding, grouting). Early detection of defects typically requires greater use of non-destructive testing techniques.

b) Minimization of in-service inspection scope by assuming that adequate measures have been taken to reduce the risk of damage, defects, or deterioration that would require major repair efforts in the future. This approach assumes that in-service inspection without the use of non-destructive testing techniques will be able to detect damage, defects, or deterioration before structural integrity is threatened. This approach requires redundancy, ductility, and robustness in the structure so that damage/defects will either propagate slowly or have little effect on overall structural reliability. This approach can be appropriate where there is conservatism in the design, rigorous quality control and inspection during fabrication, and successful historical experience from similar structures. Typical analytical procedures for quantifying robustness include linear or non-linear structural analysis coupled with redundancy (component removal) studies, strength or fatigue reliability, and coupled strength and fatigue reliability. *In situ* experience can also be used to show robustness.

Each approach is valid under different circumstances, and the choice of an inspection strategy depends on the characteristics of the owner's structures inventory, the considerations outlined in 23.3.3, and engineering judgment. Most inspection strategies are a blend of the two approaches.

A.23.4.2 Requirements

An inspection strategy considers the condition of the structure through evaluation of existing inspection data and trend analyses, together with strength and fatigue analysis results. The strategy should be sufficiently broad in scope to capture unpredictable anomalies, such as damage from dropped objects.

A data feedback mechanism is critical to maintaining the validity of the SIM process and to share lessons learned.

A.23.4.3 Inspection types

Scheduled inspections are intended to address all the requirements in 23.4.2 but should maintain flexibility to modify the scope of work if unexpected damage or deterioration is detected.

Unscheduled inspections should be performed as soon as practicable following an incident or event.

Table A.23.4-1 shows how the different inspection types address the various inspection motives.

The overall inspection strategy should recognize that the appearance of defects tends to follow a classic "bathtub" curve (i.e. clustered early in the service life, then a lull, followed by gradually increasing effects of deterioration). However, it can be difficult to determine where on the bathtub curve a structure is and how regional differences affect the curve.

A.23.4.4 Factors to consider in determining strategy

No guidance is offered.

Table A.23.4-1 — Function of inspection types

Inspection motive	Inspection type				
	Baseline	Periodic	Special	Post-event	Post-incident
Detection of degradation or deterioration	S	S	S		
Detection of fabrication defects or installation damage	S	P	P	P	
Detection of damage due to design uncertainties or errors	P	P	P	P	
Detection of damage due to environmental overload		S		P	
Detection of damage due to accidental event		S			P
Changes in function of platform or in permanent actions due to modifications		P			
Monitoring of known defects or repair effectiveness		P			
Change of ownership			P		
Reuse			P		
National or regional regulations	As required				

P primary purpose of inspection
S secondary purpose of inspection

A.23.4.5 Inspection methods and applications

The inspection strategy should consider the range of inspection techniques, the methods of deployment and the purpose of each inspection. Table A.23.4-2 illustrates how the techniques can fulfil the requirement of the different types of inspection specified in 23.4.3, while Table A.23.4-3 lists a number of inspection techniques and applicable deployment systems.

Table A.23.4-2 — Applicability of inspection tools and techniques to inspection types

Tool or technique	Inspection type				
	Baseline	Periodic	Special	Post-event	Post-incident
Air gap measurement	Initial	Trends	b	a	
Cathodic potential readings	Initial	Trends	b		
Sacrificial anode condition – visual	Existence check	Depletion check		Existence check	
Marine growth measurement		Trends	b		
Visual – without cleaning	Scour Damage Defects Debris	Scour Damage Defects Debris	b	Scour Damage Defects Debris	Damage
Flooded member detection (FMD)	Damage	Damage	a, b	Damage	Damage
Ultrasonic testing (UT)	a	a	a, b	a	a
Visual – with marine growth cleaning	a	Selected locations[a]	a, b	a	a
Non-destructive testing (NDT)	a	Selected locations[a]	a, b	a	a

[a] Used if warranted, based on evidence from inspection, analysis etc.

[b] Use depends on what is being monitored: repair performance, known defects or damage, scour sensitive areas, local corrosion, high or low cathodic potential readings, subsidence, excessive marine growth, etc.

Table A.23.4-3 — Inspection technique capabilities and deployment methods

Inspection technique	Suitability	Possible deployment methods				
		Surface use	Diving			ROV
			Air	Sat.	ADS	
Air gap measurement	Where air gap measurement devices are correctly set up, calibrated and maintained, continuous records of wave heights and tide can provide very useful information on environmental conditions. Where this can be combined with directionality data and ideally some method of estimating actions (e.g. strain gauges), the data can be used in analyses and assessment of defects and of remaining life, possibly reducing conservatisms. Satellite surveying techniques can often be used to determine levels.	X				
Marine growth recording	Marine growth comes in different forms, broadly divided into hard (generally animal, such as mussels and barnacles) and soft (seaweeds and kelps). Hard growth is generally thinner (less effective increase in member diameter) but rougher (increase in drag coefficient C_d) than soft growth. Marine growth measurements are notoriously unreliable, particularly for soft growth and for single estimates for large components. Marine growth varies with location and depth, see ISO 19901-1. Requirements for estimation of the length, type and extent of marine growth should depend on the structure's tolerance to the additional actions caused by the growth. Some structures have antifouling claddings that have performed reliably for more than 20 years.		X	X	X	X
Visual inspection without marine growth cleaning	Suitable for detection of gross damage (e.g. large deformations, severed connections, missing members), indirect signs of gross damage (e.g. gaps in or spalling of marine growth or surface coatings), and debris. When performed by ROV, visual acuity should be such that the 20/20 line can be read by the video camera system. Still camera, preferably digital, and stereo-photogrammetry provide the maximum detail and accuracy.	X	X	X	X	X
Visual inspection with marine growth cleaning	Generally used to follow-up damage identified without cleaning (corrosion, visible cracks, dents, gouges, abrasion, deformations, etc.) or targeted inspection of selected locations. Generally extent of cleaning is limited to that required for inspection. When performed by ROV, visual acuity should be such that the 20/20 line can be read by the video camera system. Still camera, preferably digital, and stereo-photogrammetry provide the maximum detail and accuracy.	X	X	X	X	X
Dimensional measurements	Usually to measure marine growth thickness, scour depth at sea floor, dent size, out-of-straightness, crack length, corrosion pit size, etc. Generally undertaken by divers using tape measures or by ROVs using scale rules and cameras or photogrammetry.	N/A	X	X	X	X
Cathodic potential readings	Measures performance of cathodic protection system. Two types of probes are available (proximity probes and contact probes). Both require calibration in the field. Proximity probes enable readings to be taken quickly and efficiently. For efficiency, evaluation of cathodic potential can often be performed with visual inspection to determine anode wastage and its interdependency with potential measurements.	N/A	X	X	X	X
Direct current measurement						

Table A.23.4-3 (*continued*)

Inspection technique	Suitability	Possible deployment methods				
		Surface use	Diving			ROV
			Air	Sat.	ADS	
FMD (flooded member detection)	Able to determine if member is flooded or not. Suitable for detecting existence of through thickness crack or other damage. Through wall damage on the leg side of connections where the leg is intentionally flooded or grout filled cannot be detected. Effectiveness depends on water depth, crack size and porosity of crack, i.e. what proportion of time is the crack open and how long it takes to flood. A crack open for only a few seconds in each storm can grow without the member flooding significantly, particularly for shallow members. Finding the cause of flooding requires further investigation with other inspection techniques. The procedure is relatively fast particularly when undertaken by ROV, thus providing an excellent tool for rapid screening of structure members for gross damage.					
	Ultrasonic technique (UT FMD) — relatively diver intense and requires accurate placing of the probes to achieve reliability. Can be used to determine the water level in members providing evidence for the cause of flooding.	N/A	X	X	X	
	Radiography (RT FMD) — Readily deployable, even from smaller ROVs. Allows for rapid coverage of many components. Source and detector mounted to U shaped frame to permit rapid and accurate placement.	N/A				X
UT (ultrasonic) — Compression wave	Used mainly for wall thickness measurements and lamination detection. The technique is straightforward and reliable for these applications. Usually performed by diver, although ROV capability exists.	X	X	X		X
UT (ultrasonic) — Shear wave, creeping wave, and time-of-flight-detection (TOFD)	Used for detecting internal volumetric indications as well as cracks and used to size indications found by other (surface) NDT techniques. Requires qualified UT inspector if performed remotely (e.g. if the probe is manipulated by divers, a qualified NDT inspector monitoring via a display screen highly improves reliability).	X	X	X		
MPI(magnetic particle inspection) (also known as MT)	Used for detection of surface breaking defects. Surface cleaning or coating removal is normally required prior to MPI application, however, cleaning to bright metal is not always necessary underwater. Different types of MPI equipment are available including articulating yokes[A.23.4-1], permanent magnets, coils and prods. Articulating yokes are considered to be the fastest and most accurate.	X	X	X	X	
ACPD (Alternating current potential drop)	Generally used for sizing defects found by other NDT methods, multiple measurements are required along the crack length, spaced (typically 5 mm – 10 mm) according to the resolution required. Requires cleaning to bare metal and trained diver.		X	X		
ACFM (alternating current field measurement)	Used to locate and size (length and depth) surface flaws. Cleaning to bright metal is not necessary, as it can operate through coatings. Training is necessary to avoid poor reliability. ACFM cannot find indications in certain geometries such as edges of gussets due to the edge effect produced from the geometry.		X	X		

Table A.23.4-3 (*continued*)

Inspection technique	Suitability	Possible deployment methods				
		Surface use	Diving[a]			ROV
			Air	Sat.	ADS	
Eddy current (also known as ET)	Used to locate and size (length) surface cracks. Cleaning to white metal is not necessary, can operate through coating. Can be used to inspect underwater welds. This requires a diver and an inspection technician above water to read the screen. Training is required.		X	X		
RT (radiographic inspection)	Used for detecting internal defects. Not routinely used for underwater inspection of offshore structures due to health and safety considerations.	X	X	X		

[a] See below for an explanation of these diving options.

The options for deployment of inspection tools and techniques should be considered in developing the inspection programme. Currently available deployment systems include the following, where the indicative diving depths can vary with regionally applied industry criteria.

a) **Surface use**

Routinely used for all surface relevant inspection techniques.

b) **Air diving**

Almost all inspection and NDT systems and tools are available in diver operable configurations. Suitably trained divers have the adaptability and dexterity to perform complex tasks and make judgments with the benefit of tactile feedback and binocular vision normally denied to ROV operators. The weight and size of inspection systems is not a major issue, as divers can rig and trim buoyancy and support as required. Most tools can be configured to operate at depth. Air divers can operate at up to 50 m (albeit that the working time decreases with depth due to decompression requirements) and have relatively simple support requirements. Surface supplied mixed gas can extend the dive depth. Diving is a hazardous occupation and divers can suffer long-term health decline. When operating in cold waters (e.g. North Sea), the complexity of the equipment increases, as hot water systems are required.

The limitations of divers are generally high cost — particularly with a DSV(diving support vessel) —and limited operational duration due to diver fatigue.

c) **Saturation diving**

Considerations are similar to those for air diving except that the divers stay at operating depth pressure for considerable times (up to 28 days), living in pressurized chambers except when working. Mixed-gas divers dive in the range of 16 m to 300 m (typically) and normally operate in saturation. The long-term health implications become greater and the options for assisting a diver in trouble are severely limited.

d) **Atmospheric diving suit (ADS)**

These “hard suit”, one-person diving systems put a human on-site but require the pilot to operate with manipulators. An ADS can have two manipulators to deploy tools. Systems are normally designed for either bottom-oriented or mid-water working. Tool interfaces are designed or adapted to suit the manipulators. ADS are normally selected for installation, maintenance, and drilling support rather than inspection programmes. Their main limitation is that the manipulator cannot always work between members with small angles.

The advantages of ADS are rapid deployment to depth, operations to 750 m and avoidance of hyperbaric exposure to the operator (pilot) and, therefore, no long-term health implications. ADS are often air transportable and suitable for deployment from some platforms. However, there are a limited number of such systems available.

e) Remotely operated vehicles (ROV)

ROV avoid the human health and safety issues associated with having divers underwater. ROV can be used to assist divers by providing additional light and cameras, but such ROV should be small enough to avoid becoming a significant hazard to the divers. Operational duration is normally unlimited other than by maintenance requirements, and depth limits can be in excess of 900 m.

Dexterity and adaptability are limited in comparison with divers, while tools for specific tasks can often be developed prior to deployment; rapid development is continuing.

The smallest ROV systems are helicopter-transportable, camera-only and camera/sonar/cathodic potential/digital-UT systems. Even these systems can be used to deploy radiological FMD equipment and other specialized systems subject to payload capacity. (ROVs are preferred for radiological FMD as radiological protection precautions are simplified.)

Larger vehicles can be equipped with tether management systems, subsea garages, manipulators, suction arms or grabs for stability at worksite, onboard hydraulic power units, marine growth removal and cleaning capabilities (high-pressure water jets, rotary wire brush). Larger ROVs can have difficulties moving within the confines of a structure and the experience of the pilot can be critical in such circumstances.

ROVs can have weld NDT (MPI, etc.) capability and remedial grinding capability.

A.23.5 Inspection programme

The inspection programme should be supplemented by an inspection specification. As a minimum, the inspection specification should

- detail anomaly thresholds for reporting or remedy (e.g. marine growth limit, cathodic potential ranges, etc.),
- specify diver and operator qualifications (see 23.8),
- specify immediate actions following discovery of an anomaly (e.g. flooded member),
- specify measurement procedures (e.g. dents, bows, holes),
- specify sensors and instrumentation,
- specify reporting styles and procedures, and
- specify required photography and video.

A.23.6 Inspection requirements

A.23.6.1 Baseline inspection

In order to facilitate monitoring of structural condition trends, the baseline inspection should establish the following for subsequent periodic inspections:

- cathodic potential measurement stations;
- scour measurement stations;
- marine growth measurement stations.

Where these data are unavailable for development of the long-term inspection strategy, penalties (e.g. unnecessary scour protection or mitigation for a structure initially installed high) can occur. Consequently, the review, retention and subsequent transfer (if required) of baseline inspection and fabrication data is extremely important.

A.23.6.2 Periodic inspection

The main mechanisms for degradation and deterioration are corrosion and fatigue. Corrosion is not generally a problem, provided the cathodic protection system is adequately designed and is maintained. Fatigue cracks can occur under cyclic actions, especially at points of stress concentration (see Clause 16). Such cracks can be found using NDT methods or by using flooded member detection (when cracks become through thickness and are sufficiently open for flooding to occur).

The extent and location of flooded member checks and NDT should be determined from consideration of structure condition, experimental data, level of consequence if defects go undetected, redundancy, strength, reliability, experience with other structures and the potential for flooding being detectable by FMD at specific locations in the event of damage. Through-wall damage on the leg (chord) side cannot be detected by FMD, for example, depending on the construction. Probabilistic methods provide an additional means to determine the inspection frequency and the location of weld inspection requirements, but can be very conservative in predicting cracks at end connections of primary members in newer structures.

Improvements in quality through familiarity and efficiencies can generally be achieved when the periodic inspection strategy is developed for a group of structures together. When the group of structures has similar characteristics and good inspection history, reduced scopes of work can generally be justified (compared to those required if the structures were considered individually). The greatest benefit is realized when the inspection intervals and scopes of work are periodically reviewed and adjusted, based on the latest inspection findings for the group of structures, as well as general industry experience.

A.23.6.3 Special inspections

Special inspections should be combined with periodic inspection scopes of work where possible. This process can be easily managed when an overall inspection strategy has been established and is being maintained.

Special inspections are used to ensure adequate performance of repairs to structural components and appurtenances by, for example, NDT of underwater wet welds, confirming the tightness of bolts in stressed ungrouted clamps, or inspection of swaging or mechanical connectors. The inspection should be performed within one year of the repair but may be undertaken during the next periodic inspection if the repairs are not critical for the structure's fitness-for-purpose.

Special inspections should also be performed to monitor growth of any known cracks at primary or safety-critical secondary member end connections.

A.23.6.4 Unscheduled inspections

Post-event inspections are used to determine the extent of damage primarily through general visual inspection. Missing members should be identified with adjacent components checked for collateral damage. The extent of obvious damage to connections should be quantified using appropriate methods. Where damage has occurred to a specific component, all connections to that component and to successive components should be inspected. Damage can sometimes be located by looking for isolated submerged areas of little or no marine growth, indicating possible overload.

Post-incident inspection should focus on areas local to the actual or possible impact location(s), e.g. inspection of members in the path of a dropped object or areas above and below water in the area of an impact. In the case of boat impact, hidden damage occurs on the underside of members when a boat is lifted by a wave or by swell.

The post event inspection strategy should

— establish a threshold for triggering inspection,

— define a nominal or default inspection scope of work (subject to modification, based on initial evaluation when an event occurs), and

— specify a method for measuring or estimating the magnitude and severity of an environmental event, based on consideration of the required accuracy and speed of provision of the information.

These items should be addressed before commencement of platform operations and should be based on an evaluation as required by 23.3.

Typical methods for estimating the magnitude or severity of an environmental event include

— in general, from observations by personnel on the platform or on nearby platforms, and from results of hindcast studies,

— for waves, from high water marks, wave gauges, ship observations,

— for wind, from anemometers,

— for an earthquake, from accelerometers, reported Richter magnitude, and distance from epicentre to platform, and

— for current, from current meters.

Structure-specific event and incident thresholds and scopes of work should be established in advance (preferably during the design), in order to avoid unnecessary inspection and enable inspection to be undertaken quickly if required. The inspection strategy should allow the flexibility to combine post-event and periodic inspection scopes of work and adjust the interval for the next periodic inspection if appropriate.

A post-incident inspection strategy includes the following important features:

— prompt and reliable reporting of incidents — owners should consider establishing protocols and notification procedures;

— early involvement of qualified personnel (see 23.8), to judge the potential significance of the incident and develop an appropriate inspection scope of work;

— close consultation with qualified personnel during offshore execution, review of findings, and assessment of need for any repairs, mitigation, future monitoring, etc.

For manned platforms, significant incidents will usually be noted and reported; however, for unmanned platforms, incidents will sometimes not be noted or reported. Thus, different strategies are often required, such as installation of sensors with automatic reporting or more frequent periodic inspections.

Inspection is more efficient and more likely to produce the data required for analysis if the SIM personnel are familiar with the structure and are able to integrate the post-incident work scope, schedule, and/or findings with other inspection activities for the structure or group of similar structures.

A.23.7 Default periodic inspection requirements

The use of a formal structural integrity management (SIM) system is recommended; however, the default inspection programme based on worldwide experience provides a predefined in-service inspection programme, should owners choose not to implement SIM.

A.23.8 Personnel qualifications

A.23.8.1 Evaluation and inspection strategy

Personnel responsible for conducting the evaluation and developing the inspection strategy should have the following qualifications.

a) Familiarity with relevant information about the specific structure(s) under consideration, including

- environmental conditions,
- design situations and criteria, structural drawings,
- structural analyses,
- fabrication and installation history,
- past inspection results (scope and findings), and
- operational history.

In the case of multiple structures, this implies familiarity with the similarities and differences among the various structures.

b) Knowledgeability in underwater corrosion processes and prevention, including

- general design principles and functional requirements for the common protection systems, and
- typical problems encountered in the field.

c) Competence in offshore structural engineering with an understanding of

- failure modes,
- likelihood of failure,
- consequences of failure, and
- different considerations for design and assessment engineering.

d) Experience in offshore inspection planning with the judgment needed to

- establish prudent and practical work scopes,
- identify components that can serve as good indicators of overall structure performance, and
- select representative components for inspection.

e) Knowledgeability in the use of inspection tools and techniques and their deployment systems, including their

- capabilities,
- limitations,
- cost,
- local availability, and
- interpretation.

f) Familiarity with general inspection findings in the offshore industry (especially for the particular geographic region) and associated experience of structural and corrosion performance.

A.23.8.2 Data collection and update

Personnel with qualifications according to A.23.8.1 should establish the requirements for content and the functional requirements for data archival, retrieval and reporting. Technicians may populate the database under the guidance of the engineer.

A.23.8.3 Inspection programme

The inspection programme contains two main elements, its specification and its execution. The qualifications required for the elements are different but complementary. Engineers with qualifications according to A.23.8.1 should

— provide advice during development of work scope and schedule,

— establish specifications for inspection tasks, and

— establish procedures for quality assurance, quality control, and data validation.

With regard to the execution of the inspection programme, the UK CSWIP[A.23.8-1] is an example of a comprehensive scheme for the examination and certification of individuals to demonstrate their knowledge and competence. The CSWIP underwater inspection committee has representatives from offshore owners, diving contractors, classification societies, and academia. The scheme covers both underwater and topsides inspection personnel. The categories of certification for underwater inspection are as follows:

a) grade 3.1U Underwater (diver) inspector (visual, cathodic protection, and ultrasonics);

b) grade 3.2U Underwater (diver) inspector (as 3.1U plus MPI, weld toe grinding);

c) grade 3.3U ROV inspector;

d) grade 3.4U Underwater inspection controller.

EN 473 [A.23.8-2] provides another example of qualification levels for those responsible for offshore inspections and for NDT inspectors.

Reference should be made to any relevant regional standards.

A.24 Assessment of existing structures

A.24.1 General

No guidance is offered.

A.24.2 Assessment process

No guidance is offered.

A.24.3 Data collection

No guidance is offered.

A.24.4 Structure assessment initiators

Inadequate deck height is an assessment initiator because of the large increase in actions associated with waves impinging on the deck. The significance of this has been demonstrated from hurricane wave and storm surge hindcast analyses of platform failures in the Gulf of Mexico and from evidence of damage due to water in the deck of surviving platforms.

Inadequate deck height can result from one or more of the following events:

— deck elevation set too low by equipment limitations at the time of installation;

— deck elevation set to clear a lower design wave height;

— inadequate wave clearance requirements at time of installation;

— field installed cellar deck;

— structure installed in water deeper than for which it was designed;

— subsidence due to reservoir compaction.

A.24.5 Acceptance criteria

A.24.5.1 Methods for determining acceptance criteria

Acceptance criteria for assessment may be developed based on the following methods.

a) Assessment with explicit probabilities of failure

The computation of explicit probabilities of structure failure, provided the failure probabilities are properly derived, and the acceptance criteria used can be satisfactorily substantiated.

b) Assessment based on risk based structural reserve strength factors (RSRs)

An example of this approach is shown in Reference [A.24.5-1] and discussed in References [A.24.5-2] to [A.24.5-9].

c) Assessment of similar structures by comparison

Design level or ultimate strength performance characteristics from an assessment of one structure may be used to infer the fitness-for-purpose of other similar structures, provided the structures' framing, foundation support, service history, structural condition and payload actions are not significantly different. If one structure's detailed performance characteristics are used to infer those of another similar structure, documentation should be developed to substantiate the use of such generic data. The following list sets out criteria that should be satisfied for structures to be considered “similar” for assessment purposes:

— the exposure level of the structure to be assessed is not L1;

— if the structure to be assessed has exposure level L2, the nearby structure that has been assessed has an exposure level of L2 or L1;

— the distance between the two structures is no more than 25 km;

— the structures are in the same water depth;

— the environmental and seismic conditions at the site of the structure to be assessed are not more severe than those at the location of the structure that has been assessed;

— the topsides arrangements of the two structures are similar and the topsides weights on the structure to be assessed are not greater than those on the structure that has been assessed;

— the structures are of the same configuration, i.e. same number of legs and same bracing pattern;

— the foundation arrangements are the same, i.e. same number and diameter of piles with the same penetration (to within 2,5 m);

— the structure to be assessed has not suffered any accidental damage;

— the materials and welding strengths and ductility on the structure to be assessed are greater than or equal to those on the structure that has been assessed —if, for any component, the strength is less than that of the equivalent component on the structure that has been assessed, a specific check of the component should be undertaken;

— the component dimensions (diameters, thicknesses and lengths) on the structure to be assessed are equal to those on the structure that has been assessed, except that thicknesses may be greater on the structure to be assessed — if, for any component, the dimensions are different to the equivalent component on the structure that has been assessed, a specific check of the component should be undertaken;

— the soil conditions at the location of the structure to be assessed are no less competent than those at the location of the structure that has been assessed;

— the ages of the structures are within 5 years of each other.

d) Assessment based on prior exposure

Prior storm exposure may be used, provided the structure has survived with no significant damage. The procedure would be to determine, either from measurements or a calibrated hindcast, the expected maximum base shear that the structure has been exposed to and then check to see if it exceeds, by an appropriate margin, the base shear required in the ultimate strength analysis check. The margin will depend on the uncertainty of the wave actions during the prior exposure, the uncertainty in structure ultimate strength, and the degree to which the structure's weakest direction was tested by the exposure actions. All sources of uncertainty, i.e. both natural variability and modelling uncertainty, should be taken into account. The margin should be substantiated by appropriate calculations to show that it meets the acceptance criteria requirements. Analogous procedures may be used to assess existing structures based on prior exposure to seismic or ice loading.

A.24.5.2 Factors for consideration with acceptance criteria

No guidance is offered.

A.24.6 Structure condition assessment

A.24.6.1 General

The quality of structural assessments is determined by the quality of the data available. The following lists summarize data that can be required.

a) General information:

1) original and current owner of the structure;

2) original and current function and use of the structure;

3) location, water depth, and orientation;

4) structure type (e.g. caisson, tripod, 4-6-8 leg);

5) number of well slots, risers, and production rate;

6) other site-specific information, manning level, etc.;

7) performance during past extreme environmental events.

b) Original design information:

1) design contractor and date of design;

2) design drawings and material specifications;

3) design code (e.g. edition of API RP2A) and design bases;

4) environmental conditions (wind, wave, current, seismic, ice, etc.);

5) deck clearance elevation (bottom of cellar deck steel);

6) operational data (permanent and variable actions and equipment arrangement);

7) soil data;

8) number, size, and design penetration of piles and conductors;

9) appurtenances (list and location as designed).

c) Construction information:

1) fabrication and installation contractors and date of installation;

2) as-built drawings;

3) fabrication, welding and construction specifications;

4) material traceability records;

5) pile and conductor driving records;

6) pile grouting records, if applicable.

d) Information on structure history:

1) environmental action history (e.g. tropical cyclones, seismic events);

2) operational action history (e.g. collisions and other accidental actions);

3) survey and maintenance records;

4) repairs (descriptions, analyses, drawings and dates);

5) modifications (descriptions, analyses, drawings and dates).

e) Information on present condition:

1) for all decks, the actual size, location and elevation;

2) for all decks, the existing permanent and variable actions and equipment arrangement;

3) field measured deck clearance elevation (bottom of steel);

4) production and storage inventory;

5) appurtenances (current list, sizes and locations);

6) wells (number, size and locations of existing conductors);

7) recent “above water” inspection (Level I or equivalent, see Clause 23);

8) recent underwater structure inspection (Level II, minimum, or equivalent, see Clause 23).

If original design data or as-built drawings are not available, assessment data can be obtained by field measurements of dimensions and sizes of important structural members and appurtenances. The thickness of tubular members can be determined by ultrasonic procedures, both above and below water, for all members except the piles. If the wall thickness and penetration of the piles cannot be determined and the foundation is a critical element in assessing the adequacy of the structure, it will generally not be possible to perform an assessment. In this case, it can be necessary to downgrade the use of the structure to a lower exposure level, see 6.6, by reducing the risk or to demonstrate adequacy by prior exposure.

A.24.6.2 Topsides surveys

No guidance is offered.

A.24.6.3 Underwater and splash zone surveys

No guidance is offered.

A.24.6.4 Foundation data

Many sampling techniques and laboratory testing procedures have been used over the years to develop soil strength parameters. With good engineering judgment, parameters developed with earlier techniques can be upgraded based on published correlations. For example, design undrained shear strength profiles developed for many structures installed prior to the 1970s were based on unconfined compression tests using 57,2 mm (2,25 in) diameter driven wireline samples. Generally speaking, unconfined compression tests give lower strength values and greater scatter than unconsolidated undrained compression tests, which are now considered the standard. Studies have also shown that a 57,2 mm (2,25 in) sampler produces greater disturbance than the 76,2 mm (3,0 in) diameter thin-walled push samplers now typically used offshore. Therefore, depending on the type of sampling and testing associated with the available data, it can be appropriate to adjust the undrained shear strength values accordingly.

Pile driving data should be used to provide additional insight into the soil profiles at each pile location and infer the elevations of pile end bearing strata.

Actual pile lengths from driving records should be used, when available. Otherwise, lengths should be conservatively assessed. The effects of global seabed scour, local scour and/or potential loss of soil-pile contact should be taken into account.

A.24.7 Actions assessment

A.24.7.1 General

No guidance is offered.

A.24.7.2 Metocean parameters and environmental actions

ISO 19901-1 and Clause 9 contain requirements and guidance on metocean parameters and the environmental actions resulting from them.

A.24.7.3 Deck elevation and additional environmental actions

A simplified silhouette method for estimating the global wave/current action affecting a structure where the bottom of the cellar deck can be below the top of the waves is described below. The procedure is calibrated to measurements for quite simple structures in wave tank tests in which tropical cyclone and winter storm waves were modelled. Alternative methods can be more appropriate for more complex decks or when the local actions on the deck as the wave passes through the structure are to be taken into account.

The result of applying this procedure is the magnitude and point of application of the horizontal action on the topsides for a given wave direction. The variability of the action on the topsides for a given wave height is rather large. The coefficient of variation (COV) determined from the specific tests is approximately 0,35. The action on the topsides should be added to the associated environmental action on the structure.

Other calculation procedures for wave/current actions on topsides for static and/or dynamic analyses can be used, and should be validated with reliable and appropriate measurements of global wave/current actions, either in the laboratory or in the field. The water particle flow direction has vertical as well as horizontal components so for a large deck with H-beams more complex actions are generated than captured in the simplified silhouette model as waves pass through the structure. A short duration of the wave in a deck event can also generate considerable dynamic effects.

The simplified procedure requires a calculated crest height and crest particle velocity. The crest height should be calculated from the wave height, associated wave period, and storm tide for the ultimate limit state analysis (ULS) (see Clause 9 and ISO 19901-1). The actions on the structure are calculated using the same wave parameters (height, period, direction, position and theory). For more accurate analyses, specific consideration of the selection of wave kinematics should be ensured.

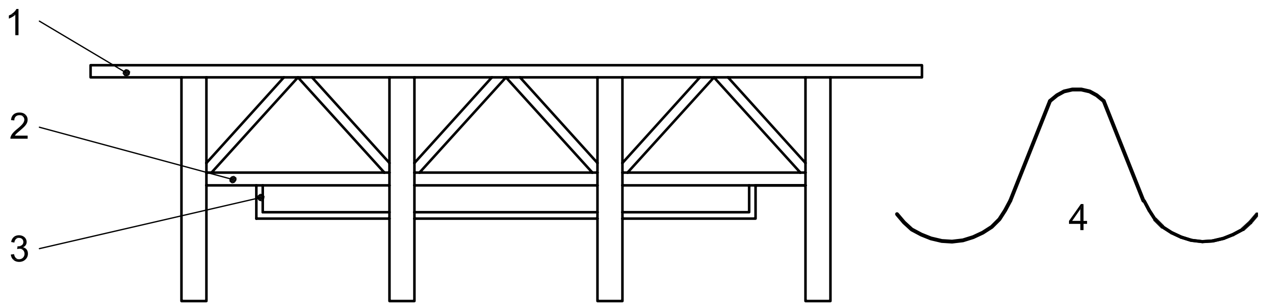
EXAMPLE Freak waves, which can be steeper than predicted by the Stokes 5th order wave theory, have high crest velocities and which, in combination with large deck areas, can generate significant actions.

The calculation of actions from waves impinging on the deck of an offshore structure is an area of ongoing research. The phenomena are complex and influenced by factors including the type of waves, level of inundation, deck configuration, wave phase and deck proportions. The simplified silhouette method should only be applied when it can be shown to be appropriate to the case being assessed.

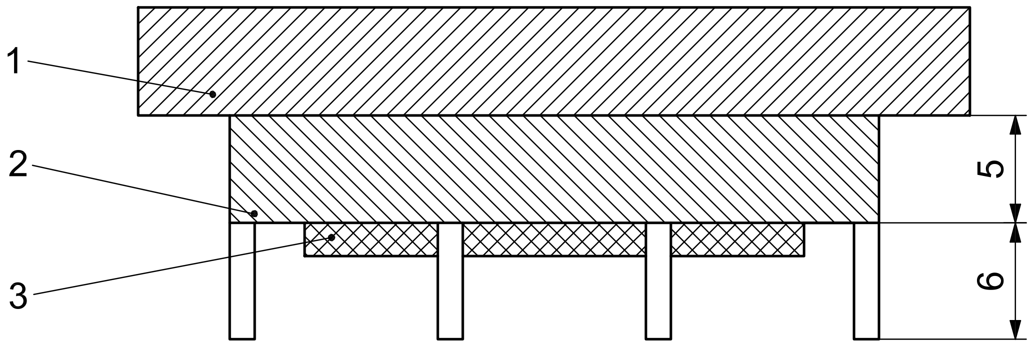
The steps for computing the action on the topsides and its effective elevation using the simplified silhouette method are as follows:

a) From the crest height, compute the area of the wetted silhouette, A, projected in the wave direction, θ_w.

   The full silhouette area for a topsides is defined as the shaded area in Figure A.24.7-1, i.e. the area between the bottom of any scaffold deck and the top of the “solid” equipment on the main deck. The silhouette area used for action calculations on topsides, A, is a subset of the full area, extending up to an elevation above mean sea level that is equal to the sum of the tide, storm surge and crest height required for the ULS analysis, as determined in Clause 9.



a) Deck structure arrangement



b) Silhouette areas

Key

1 main deck

2 cellar deck (elevation of underside of cellar deck is used for inadequate deck height trigger)

3 scaffold deck

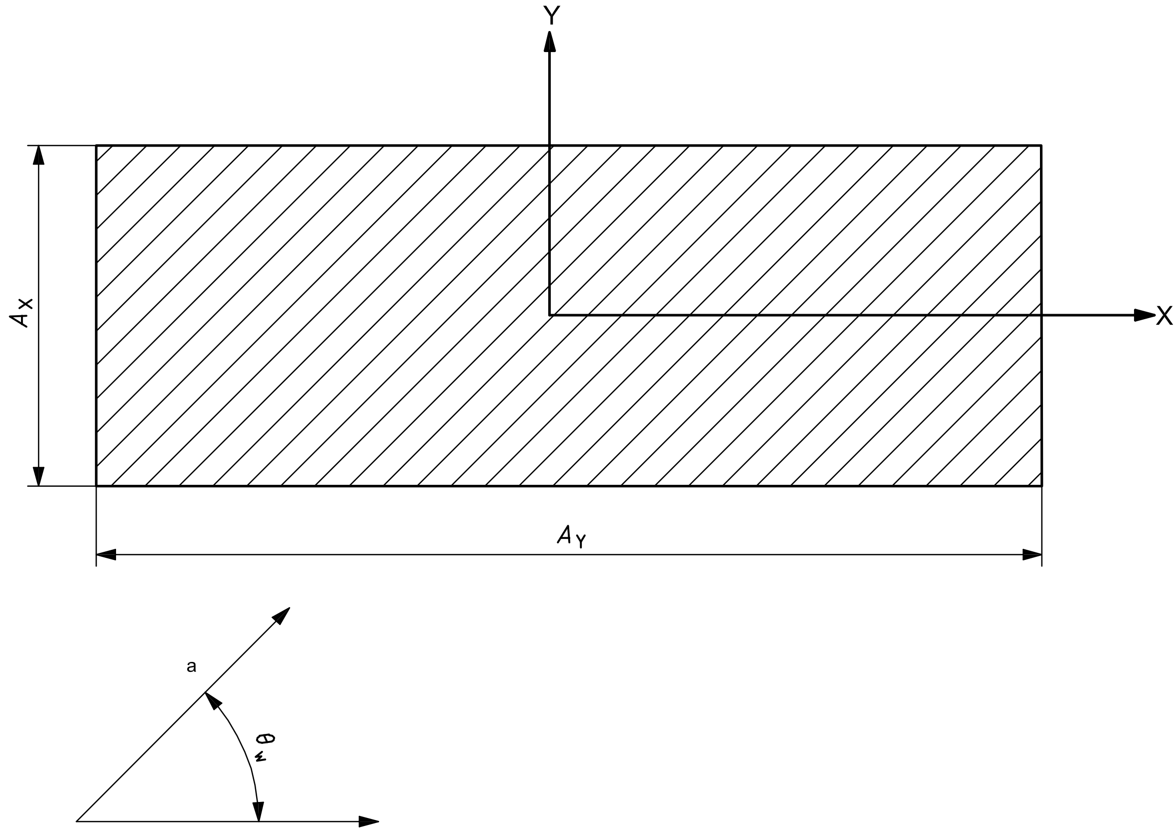
4 wave

5 deck legs and braces are part of the topsides area

6 deck legs and braces are part of the structure

Figure A.24.7-1 — Silhouette area definition (view of topsides elevation)

For lightly framed sub-cellar deck sections with no equipment, such as a scaffold deck consisting of angle iron, one-half of the silhouette area for that portion of the full area may be used. The areas of the deck legs and bracing above the cellar deck are part of the silhouette area. Deck legs and bracing members below the bottom of the cellar deck should be modelled along with structural members in the calculation procedure for actions on the structure, see Clause 9. Lattice structures extending above the solid equipment on the main deck can be ignored in the silhouette. See Figure A.24.7-2.



Key

θ_w wave direction measured from X-axis

A_x silhouette area perpendicular to X-axis

A_y silhouette area perpendicular to Y-axis

A silhouette area perpendicular to wave = $A_x \cos\theta_w + A_y \sin\theta_w$

[a] Wave heading.

Figure A.24.7-2 — Wave direction and silhouette area projection

b) Use the appropriate wave theory (ISO 19901-1), as recommended in 9.6, and calculate the maximum wave-induced horizontal fluid velocity in the direction of the wave, U_w, at the crest elevation or the top of the main deck silhouette, whichever is lower. In the event that the crest elevation is above the top of the main deck silhouette, the velocity should be recalculated assuming the tidal level that will the locate the crest elevation as close to the top of the silhouette as possible.

c) The representative wave/current action on the topsides, $E_{topsides}$, is computed using Equation (A.24.6-2):

$$E_{topsides} = \frac{1}{2}\rho_w \, C_d \left(\alpha_{wk} \, U_w + \alpha_{cb} \, U_c\right)^2 A \tag{A.24.6-2}$$

where

ρ_w is the density of sea water;

U_w is the fluid velocity from step 2;

U_c is the current speed in line with the wave;

α_{wk} is a wave kinematics factor (0,88 for tropical cyclones and 1,0 for winter storms);

α_{cb} is the current blockage factor for the structure.

The drag coefficients, C_d, are given in Table A.24.7-1 and have been determined from model tests. Judgment is required to determine suitable values for other values of θ_w.

Table A.24.7-1 — Drag coefficient C_d for wave/current actions on topsides

Deck type	C_d end-on and broadside	C_d $\theta_w = 45°$
Heavily equipped (solid)	2,5	1,9
Moderately equipped	2,0	1,5
Bare (no equipment)	1,6	1,2

d) The action $E_{topsides}$ should be applied at mid-height of the wetted silhouette area, i.e. at 50 % of the distance between the lowest point of the silhouette area (see Figure A.24.7-1) and the lower of either the wave crest or the top of the silhouette area.

A.24.7.4 Seismic design considerations

No guidance is offered.

A.24.7.5 Ice conditions and actions due to ice

No guidance is offered.

A.24.8 Screening assessment

No guidance is offered.

A.24.9 Resistance assessment

A.24.9.1 General

No guidance is offered.

A.24.9.2 Design level analysis procedure

A.24.9.2.1 General

The values of actions may be refined if there is sufficient information available to justify such refinement. Examples of refinement of actions include the following:

— more detailed review and application of permanent and variable actions, in particular in identifying variable actions that cannot occur simultaneously (such as maximum rig set-back and maximum rig hook load), or that can be controlled by operating practice (e.g. controls on lay-down area use);

— exploitation of directionality of environmental conditions where this has not been taken into account previously;

— reduced environmental parameters for L2 and L3 structures, where reduced partial action factors, reduced return periods or factors on environmental parameters are provided in Annex H for certain geographic areas.

A.24.9.2.2 Structural steel design

A.24.9.2.2.1 Members

Refined member checks may be performed on a case-by-case basis, with consideration given to any inherently conservative assumptions contained within the design requirements of Clauses 13 to 15 and 17.

The strength of a member can be refined by considering

a) effective length factors (K),

b) moment reduction factors (C_m),

c) use of face to face lengths, and

d) moment relief due to joint flexibility.

Care should be taken when applying refined component checks to ensure consistency between analysis and utilization check assumptions.

In addition, it is noted that computer-derived utilization checks often use simplifying assumptions, which although satisfactory for design, can lead to undue conservatism in assessment. Therefore, the analyst should be conversant with the detailed background to the underlying assumptions and routines adopted by the software to eliminate this source of conservatism.

Refined component checks are likely to include one of the following methods:

— hand calculations;

— computer spreadsheet calculations;

— computer calculations using mathematical software;

— automatic computer code checks using more appropriate parameters.

Ongoing research, if used to determine the strength of members, should be carefully evaluated to ensure applicability to the type of member, its internal forces, and the level of confidence in the conclusions of the research. For example, while the use of smaller values for effective length (K) factors can be appropriate for members developing large end moments and high levels of internal forces, these smaller (K) factors are not necessarily appropriate for lower levels of internal forces.

Because of availability and other non-structural reasons, members can be made from steel with yield strength higher than the specified minimum yield strength. If no reliable data exist, tests may be used to determine the material grades used so that appropriate yield strengths can be used.

A.24.9.2.2.2 Connections

Joints are usually assumed to be rigid in the global structural model. Significant redistribution of member forces can result if joint flexibility is included in the analysis, especially for short members with small length to depth ratios and for large leg can diameters where skirt piles are used. Joint flexibility analysis should use finite element methods or recognized analytical expressions, as appropriate. Steel joints can have higher strength than currently taken into account in design. Similarly, the strength of grouted joints as well as grout stiffness and strength can have higher values than normally used for design (see Clause 15). Such effects may be utilized in assessment when properly justified and documented.

A.24.9.2.2.3 Fatigue

The assessment shows adequate fatigue durability if one of the following criteria is met:

— the results of a fatigue assessment in accordance with Clause 16 shows that the fatigue lives of all members and joints are at least equal to the total design service life, and the inspection history shows no fatigue cracks or unexplained damage;

— a fatigue assessment in accordance with Clause 16 has identified the joints with the lowest fatigue lives and periodic inspection of these joints finds no fatigue cracks or unexplained damage;

— where fatigue lives of any members and joints are calculated to be less than the total design service life of the structure and fatigue damage has been identified, the structure may be assumed to be fit-for-purpose, provided conservative fracture mechanics predictions of fatigue crack growth demonstrate adequate future life and periodic inspection monitors crack growth of the members or joints concerned.

A.24.9.2.3 Foundations

No guidance is offered.

A.24.9.2.4 Fitness for purpose

A design level analysis as described in 24.9.2 may result in components being assessed as being over-utilized. A traditionally framed fixed steel offshore structure generally contains a degree of redundancy in that there are components that can suffer overstress and allow internal forces to be redistributed within the remainder of the structure, such that the structure still retains adequate safety. In these circumstances, the structure may be assessed using the linear elastic redundancy analysis procedure outlined below. All appropriate partial action and resistance factors for design given in Clauses 12 to 15 and 17 should be adopted. Refined component strength checks as described above may also be used.

In a linear elastic redundancy analysis, the structure is analysed using the methods in 12.5.2 for quasi-static linear analysis. Any members or joints which do not satisfy the utilization checks described in Clauses 13 to 15 and in Clause 17 are removed from the linear elastic model. The internal resisting forces and moments associated with full utilization of the component should be applied at the end nodes. The structure should then be subjected to a further analysis to demonstrate that redistribution of internal forces can be accommodated without causing further components to exceed a utilization of 1,0. The procedure can be repeated, progressively removing members if they exceed a utilization of 1,0. However, this type of analysis is not recommended where there are multiple failures of individual components, multiple failure paths or where design analysis component utilization exceeds 1,10 (i.e. 10 % over-utilized), as the linear elastic assumptions, implicit within the analysis, can become invalid.

The process is repeated for each storm direction; however, components that are disabled due to a storm in one direction may be assumed to be intact when considering storms from another direction.

If all critical components within the structure are assessed to have utilization less than unity after member removal analysis, and global deformations are acceptable, then the structure can be considered fit-for-purpose, and no further analysis is required. However, prevention and mitigation measures should also be considered by the owner, as the level of structural reliability will be inherently less than for a structure complying with the design requirements given in Clauses 13 to 15 and Clause 17.

If load redistribution cannot be adequately demonstrated by means of linear elastic member removal analysis, a non-linear ultimate strength analysis, as described in 24.9.3 should be performed.

A.24.9.3 Ultimate strength analysis procedure

Reference [A.24.9-1] provides information on conducting non-linear ultimate strength analysis. An example of using risk-based reserve strength ratios (RSRs) is shown in Reference [A. 24.5-1] and discussed in References [A.24.5-2] to [A.24.5-9]. RSR values for certain regions are discussed in A.9.9.3.3 and in References [A.24.9-2] and [A.24.9-3].

At the discretion of the owner, structural reliability analysis (SRA) may be performed to determine the ultimate system strength of a structure. SRA depends on the accuracy of statistics associated with many parameters, including the wave climate and the actions resulting from that environment, as well as with the resistances of components and structural systems. The use of SRA requires extreme care and there is insufficient knowledge of the statistics to enable requirements or recommendations to be included in a standard. Acceptance is highly dependent on the knowledge and skill of the analyst and the data upon which the analysis is based. It is recommended that thorough validation of the techniques and application of those techniques be undertaken, that acceptance criteria be agreed upon between the regulator (where one exists) and the owner, recognizing the scope for over optimism in determining the RSR, and that the results of the SRA be combined with consideration of the costs and benefits of strengthening or other remedial measures.

A.24.10 Prevention and mitigation

Reference [A.24.10-1] provides examples of mitigation methods. These can include removal of topsides equipment and parts of the structure that are no longer necessary, such as pile driving guides and redundant conductors, risers and caissons.

A.25 Structure reuse

A.25.1 General

No guidance is offered.

A.25.2 Fatigue considerations for reused structures

Remedial measures such as grinding welds, grouting and reinforcing can improve fatigue performance and can be beneficial when a structure is to be reused.

A.25.3 Steel in reused structures

No guidance is offered.

A.25.4 Inspection of structures to be reused

A.25.4.1 General

No guidance is offered.

A.25.4.2 Initial condition assessment of structural members and connections

No guidance is offered.

A.25.4.3 Extent of weld inspection

As the costs and safety implications of inspection are very much lower when the structure is out of the water, it is recommended that inspection out of the water is undertaken for structures to be reused.

The following describes the minimum recommended inspection extent, but this should be modified in light of the structural assessment for the reuse condition and the previous in-service inspection history.

When only partial testing is required for welds in an area, the weld testing should be distributed such that the most critical components are included in the inspection, and such that areas of welds most susceptible to weld defects or to damage from previous service are examined.

a) **Primary tubular members**

The extent of inspection of primary structure should be determined by comparing the design actions and stresses (including removal and reinstallation actions and stresses) for the new site with those to which the welds previously have been designed for and/or exposed to.

Where new design actions and stresses are less than or equal to initial design or actual actions, then the extent of inspection, if any, should be determined based on NDT documentation or the results of the initial spot survey.

Where new design actions and stresses are significantly greater than initial design or actual actions and stresses, or when comparison based on initial design or actual actions (stresses) is not possible, a minimum of one bracing member and one leg spanning between each level should be inspected.

Additional inspection should be performed where in-service damage is known or suspected.

Tubular member inspection should be done using ultrasonic testing (UT) or magnetic particle inspection (MPI).

b) **Primary tubular joints**

The brace-to-chord and brace-to-stub welds of all tubular joints with high static or fatigue utilizations or prone to accidental damage should be 100 % inspected. Where NDT inspection of these connections reveals significant defects, additional inspection of other connections should also be performed.

A minimum of one brace to chord connection at each level and one X-brace connection between levels, as applicable, should be 100 % inspected.

Tubular joint inspection should be done using UT or MPI. For tubular connections not having Class CV2Z or better steel in the heavy wall joint-cans both UT and MPI should be performed.

c) **Piles**

A minimum of 10 % of each individual longitudinal and circumferential seam on piles to be reused should be inspected using UT or MPI.

100 % of each individual field splice in piles should be inspected using UT or radiography (RT).

d) **Non-redundant bracing and subassemblies**

Non-redundant bracing and subassemblies (e.g. padeyes, lifting bracing, single level conductor guide framing level above sea floor) should be 100 % inspected using UT or MPI.

Attachment welds connecting non-redundant bracing/subassemblies to main members should be 100 % inspected using UT or MPI.

e) **Redundant bracing and subassemblies**

A minimum of 10 % of each individual weld on redundant bracing and subassemblies (e.g. multi-level conductor guide framing, secondary splash zone and sea floor bracing, boat landings) should be visually inspected.

A minimum of 10 % of each individual attachment weld connecting redundant bracing/subassemblies to main members should be visually inspected.

f) **Deck members and connections**

The requirements for reuse of topsides structures are given in ISO 19901-1.

A.25.4.4 Corrosion protection systems

No guidance is offered.

A.25.4.5 Inspections for removal of structures from prior site

No guidance is offered.

A.25.5 Removal and reinstallation

Some particular considerations for reuse that do not apply to new structures include the following.

a) **Removal**

Structures which cannot be lifted onto barges can be removed by controlled de-ballasting and skidding the structure back onto a properly configured launch barge. Such operations require precise control of barge ballasting, positioning and alignment between structure and barge. Environmental conditions for such operations can be more restrictive than usual for installation.

b) **Buoyancy and re-floating**

When removal from a prior site requires re-floating of platform components, such as the structure, additional buoyancy can be required in excess of that provided when the structure was originally installed to compensate for loss of buoyancy and for additional weights not present during the original installation, e.g. grouted piling.

c) **Marine growth removal**

When removing structures for reuse, appropriate equipment for marine growth removal from sea fastening locations should be provided. If the structure is to be skidded back onto a launch barge, marine growth should be removed from launch rails to ensure reasonable predictions of the coefficient of friction and the sling forces on padeyes and winches. Water blasting or sand blasting to remove marine growth has been found effective.

A.25.6 In-service inspection and structural integrity management

No guidance is offered.

Annex B
(informative)

CTOD testing procedures

B.1 Testing procedure requirements

When required by 20.2.2.5, CTOD testing shall be conducted in accordance with the CTOD-related clauses of ISO 12135. Annex B provides supplementary requirements, based on a specimen geometry as shown in Figure B.1.

NOTE Additional alternative guidance can be found in BS 7448[B.1] or ASTM E 1290-02[B. 2].

B.2 Test-assembly welding

The test assembly shall be prepared as shown in Figure B.1. The square bevel shall be machined as square as is possible. The following requirements also apply.

a) Back-gouging shall be carefully controlled, gouging into the square bevel shall be avoided. Careful grinding shall be used to achieve the final groove profile.

b) Surfaces shall be machined to obtain as large a thickness of the test specimen, B, as is practical for the test weld.

c) Mechanized welds in the 1G position may be made with the test plate assembly rotated 15° to 20° about the weld axis so that the angle of attack between the electrode and the edge preparation, the degree of weld bead overlap and the clearance between the electrode tip and the edge preparation simulate the production welding arrangement. Temporary flux dams may be attached for SAW. For welding the cap passes, the test plate assembly may be restored to its flat position.

B.3 Number and location of CTOD specimens

CTOD specimens shall be located within the weld metal (and the HAZ if required) such that the notch positions are in accordance with Figure B.2.

Three valid weld metal specimens from the highest heat input/interpass temperature and three valid specimens from the lowest heat input/interpass temperature to be qualified shall be obtained for each welding procedure.

When the pre-production steel qualification requirements of 20.2.2.5.2 have not been met, the HAZ specimens shall be obtained from the highest heat input/interpass temperature and from the lowest heat input/interpass temperature. Twelve specimens shall be prepared from cross-section B in Figure B.2, and 5 each from cross-sections C and D in Figure B.2.

The three “cross-section B” specimens with the lowest CTOD values from each of the high and low heat input sets shall be sectioned as shown in Figure B.3, to characterize the fracture initiation site.

B.4 Specimen preparation

The specimen shall be prepared as follows:

— for plate thickness $T < 75$ mm (3 in), the specimen width shall be $W = 2\ B$; for $T > 75$ mm (3 in), $W = B$, where T, W and B have the meaning shown in Figure B.1;

— the machined groove, saw cut or electron discharge machine (EDM) notch, and fatigue cracking shall comply with the selected CTOD test procedure.

The procedure for determining notch locations is shown in Figure B.2 and is as follows:

a) locate the thickest pass that is both in the central 2/3 of the specimen thickness and immediately adjacent to the non-bevelled side of the weld;

b) locate line A (centre line) which lies through the centre of this pass;

c) locate the reference line R, perpendicular to the specimen surface and dividing the fusion line such that over the middle 80 % of the specimen width, equal portions of the fusion line are to each side of the reference line;

d) locate lines B (fusion line + 0,4 mm), C (HAZ + 2 mm), and D (HAZ + 5 mm), at 0,4 mm, 2 mm, and 5 mm from the reference line.

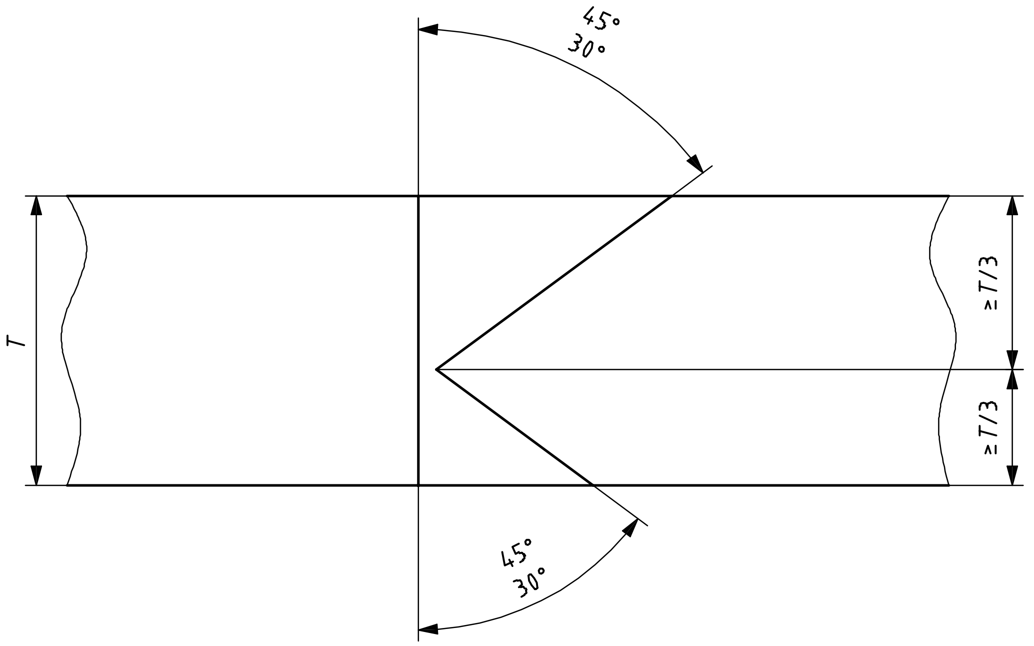
Surface-notched subsidiary-geometry specimens with $B = W$ shall be used for repair weld tests unless the repair weld consumable has been qualified by CTOD testing of a full penetration butt weld made using the same welding parameters in the same position.

B.5 Pre-compression

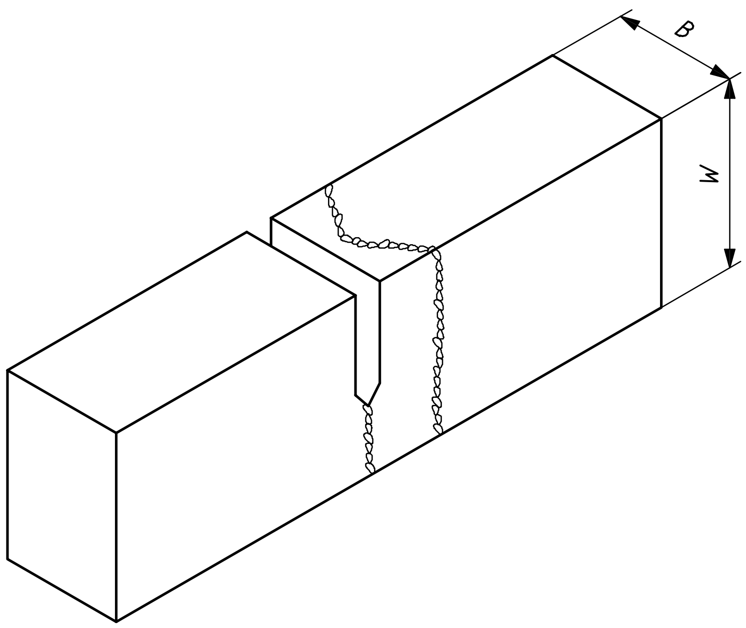
For specimens in the as-welded condition, a pre-compression dimple shall be applied to both sides of the specimen at the tip of the machined notch. The depth of the dimple shall not exceed 0,5 % strain per side (i.e. 0,01 B total). After the first series of specimens have been tested, this amount may be reduced in order to achieve a more uniform fatigue pre-crack front. The dimple shall be made with hardened indenters with a minimum diameter of 19 mm (3/4 in). Multiple overlapping indentations may be used. The location of the dimple may overlap the machined notch. However, it shall be at least 6,4 mm (1/4 in) from the side of the specimen opposite from the machined notch (to prevent distorting the back surface).

B.6 Sectioning

Specimen sectioning and examination shall be in accordance with Figure B.3.



a) Weld preparation



b) CTOD specimen

Key

B specimen thickness

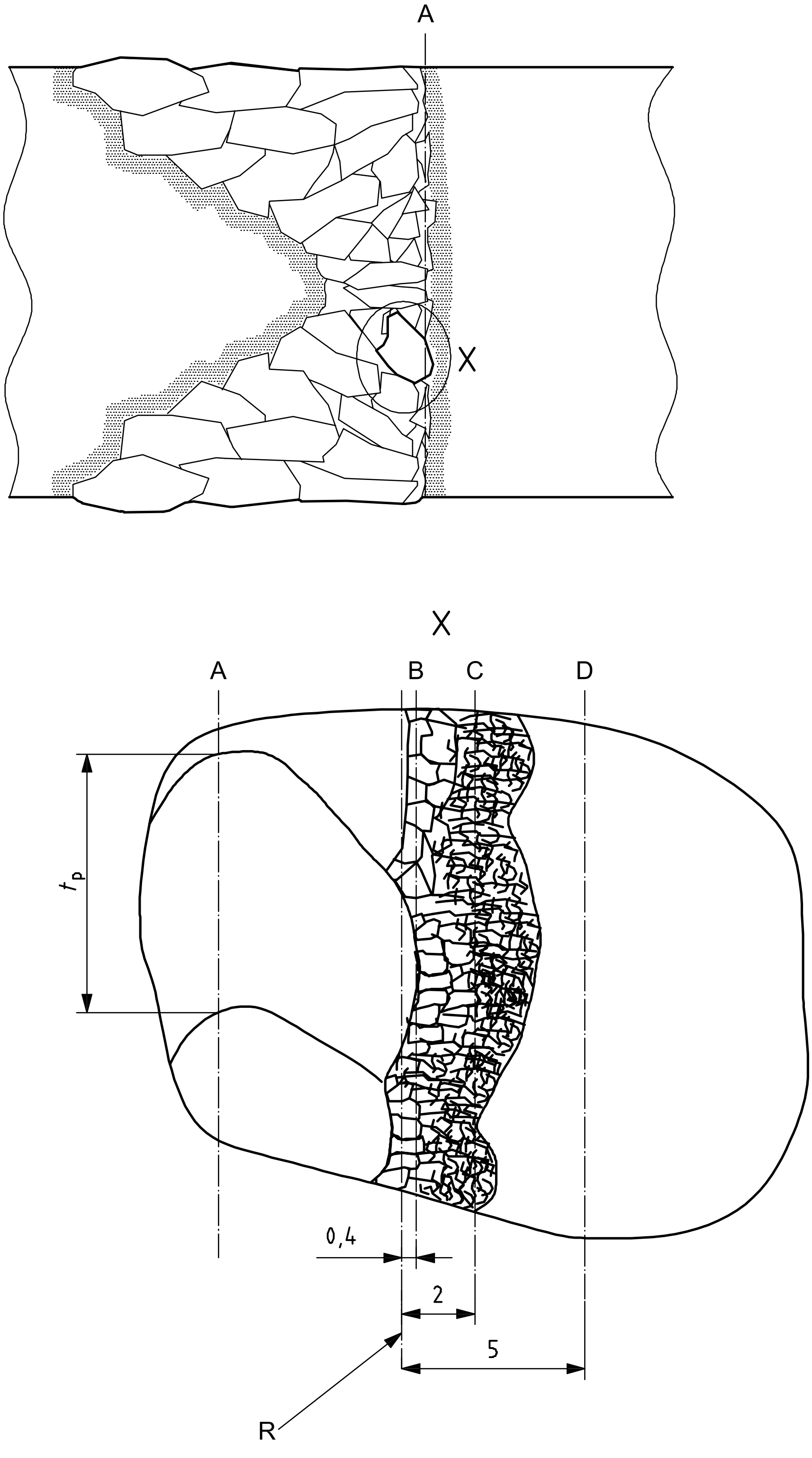
T plate thickness

W specimen width

NOTE See B.4.

Figure B.1 — CTOD specimen preparation

Dimensions in millimetres

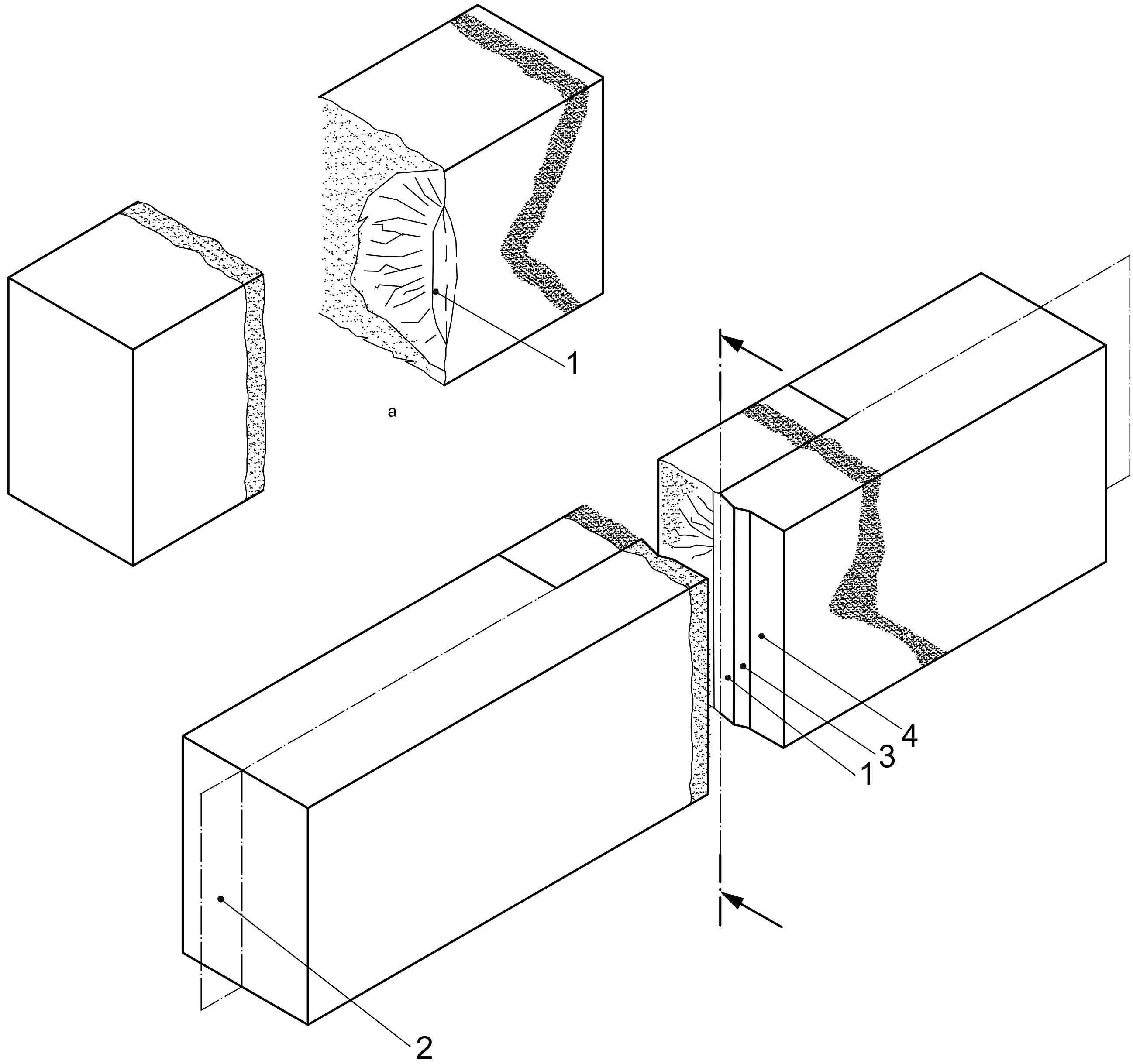


Key

A centre line of thickest weld pass
B fusion line + 0,4 mm — coarse grain HAZ
C HAZ + 2 mm — fine grain HAZ
D HAZ + 5 mm — subcritically reheated base metal
t_p thickness of thickest weld pass in middle 2/3 of weldment
R reference line, see B.4 c)

NOTE See B.4.

Figure B.2 — Notch locations for CTOD specimens



Remove a full-thickness section from each side of the fracture. Polish and etch on the plane of examination. Examine fracture features in relation to the weld beads and other artefacts.

Key

1 fatigue crack front

2 plane through specimen

3 saw cut

4 machined notch

[a] Plane of examination.

Figure B.3 — CTOD specimen sectioning and examination

Annex C
(informative)

Material category approach

C.1 Selection of material category (MC)

The responsibility for selecting an appropriate material category for the structure belongs to the owner. This decision can be based on the guidelines below.

- MC1 — This category is generally used for exposure level L1 structures. It was modelled on advanced fracture control methods used for world-record fixed platforms and compliant towers.
- MC2 — This category is generally used for exposure level L2 structures. It was modelled on common practices for major structures in moderate water depths.
- MC3 — This category is generally used for exposure level L3 structures. It was modelled on common practices for minor (but structurally redundant) platforms in shallow water.

C.2 Selection of toughness class

The required toughness class is determined from Table C.1, based on

- the type of component,
- the material category (MC) of the structure, and
- the steel group, in turn based on SMYS, where the required specific SMYS has previously been determined, along with thickness, by design analysis and calculation.

C.3 Specific steel selection

Specific material selection shall be shown in design drawings and specifications.

The strength groups and toughness classes are used to reference welding requirements, e.g. preheat and electrode selection, where these tend to follow the steel. Plates, sections and rolled tubular sections specified by the designer should conform to a recognized specification. Supplementary specifications can be required for steel used to fabricate items intended for particularly arduous duty or where recognized steel grades are used in a thickness that is above the stated limits.

Table C.2 shows a correlation of steel group and toughness class for steel plates to US specifications. Tables C.3 and C.4 do the same for wide flange and tubular members. Steels that are not included in the lists given in C.1 and C.2 above may be used, as long as they satisfy the provisions of Clauses 19 and 20, with particular attention to weldability (see also A.19.2.2.5).

Table C.1 — Material category - material selection for the structure

Component location in the structure		SMYS group [e]	Toughness class		
			MC1	MC2	MC3
Joint cans [a]	Up to 50 mm (2 in) thick	II	CV2Z	CV2Z	CV2 [b]
	Greater than 50 mm (2 in) thick	II	CV2ZX [c]	CV2Z [c]	CV2 [b]
Bracing	Primary [d]	I	NT	NT	NT
		II	CV2 or CV1	CV1	—
		III	CV2	—	—
	Secondary	I	NT	NT	NT
	Brace end stubs at nodes	I	CV2	—	—
		II	CV2	—	—
Legs	Launch legs	I	—	—	NT
		II	CV2	CV2 or CV1	—
		III	CV2	—	—
	Launch rail	I	—	—	NT
		II	CV2	CV1	—
		III	CV2	—	—
	Legs elsewhere	I	CV1	CV1 or NT	NT
		II	CV2 or CV1	CV1	—
		III	CV2	—	—
Stiffeners	At nodes	II	CV2	CV2	CV2
	Elsewhere	I	NT	NT	NT
		II	CV1	NT	—
Piling	Heavy wall at sea floor	II	CV2	CV2 or CV1	CV1
	Elsewhere	I	NT	NT	NT
		II	CV1	—	—
	Compliant tower	III	CV2	N/A	N/A
Padeyes	Major lifts, padeye greater than 50 mm (2 in) thick	II	CV2ZX	CV2Z	CV2Z
	Major lifts, padeye 50 mm (2 in) thick or less	II	CV2Z	CV2Z	CV2Z
Miscellaneous	Conductor panels	I	NT	NT	NT
	Boat landings, walkways	I	NT	NT	NT

— indicates that this combination is not normally used
N/A not applicable

[a] Includes joint cans in legs and in primary bracing.
[b] Specify steel with a low sulphur content, below 0,006 %.
[c] Consider CV2ZX for nodes greater than 75 mm (3 in) thick.
[d] A higher toughness class is recommended for thicker primary bracing, especially in areas subject to collisions.
[e] SMYS groups IV and V have had limited historical use, and are therefore omitted from this table. Selection of toughness class is left to the designer.

Table C.2 — Correlation of steel group and toughness class for steel plates to US specifications

Steel group	Toughness class	Specification	Grade	Thickness mm (in)	SMYS MPa (ksi)	Comments and recommended supplements a, b, c, d, e
I	NT	ASTM A36[C.1]		⩽ 50 (⩽ 2)	250 (36)	A (T > 19 mm (0,75 in), B
		ASTM A131[C.2]	A	⩽ 12 (⩽ 1/2)	235 (34)	
	CV1	ASTM A131[C.2]	D		235 (34)	
		ASTM A516[C.3]	65		240 (35)	B, C (Charpy at 5 °C (40 °F)
		ASTM A573[C.4]	65		240 (35)	B, C (Charpy at 5 °C (40 °F)
		ASTM A709[C.5]	36T Zone 2		250 (36)	A, B (P_{CE} ⩽ 0,40), C (per A709 supplement S83)
	CV2	ASTM A131[C.2]	E		235 (34)	C (per A131 supplement S86), D
II	NT	ASTM A572[C.6]	42	⩽ 50 (⩽ 2)	290 (42)	A [T > 12 mm (0,5 in)], B (P_{CE} ⩽ 0,43), P_V ⩽ 0,10
		ASTM A572[C.6]	50	⩽ 50 (⩽ 2)	345 (50)	A [T > 12 mm (05 in)], B, P_V ⩽ 0,10
	CV1	API Spec 2MT1[C.7]			345 (50)	
		ASTM A709[C.5]	50T Zone 2		345 (50)	Gulf of Mexico: A, B, C (per A709 supplement S83)
		ASTM A709[C.5]	50F Zone 3		345 (50)	Cold, non Arctic: A, B, C (per A709 supplement S84)
		ASTM A131[C.2]	AH36		360 (51)	Optional A131 supplement S87 (normalize T > 19 mm (0,75 in)
	CV2	API Spec 2H[C.8]	42		290 (42)	D
		API Spec 2H[C.8]	50	⩽ 65 (⩽ 21/2)	345 (50)	D
		API Spec 2H[C.8]	50	> 65 (> 21/2)	324 (47)	D
		API Spec 2W[C.9]	42		290 (42)	
		API Spec 2W[C.9]	50		345 (50)	
		API Spec 2W[C.9]	50T		345 (50)	Minimum UTS = 75 MPA (517 ksi)
		API Spec 2Y[C.10]	42		290 (42)	
		API Spec 2Y[C.10]	50		345 (50)	
		API Spec 2Y[C.10]	50T		345 (50)	Minimum UTS = 75 MPA (517 ksi)
		ASTM A131[C.2]	EH36		360 (51)	C (per A131 supplement S86), D
		ASTM A537[C.11]	Class 1	⩽ 65 (⩽ 21/2)	345 (50)	B, C (Charpy at –40 °C (–40 °F), D
		ASTM A633[C.12]	C, D		345 (50)	B, C (Charpy at –40 °C (-40 °F), D
		ASTM A678[C.13]	A		345 (50)	B, C (Charpy at –40 °C (–40 °F), D

Table C.2 (*continued*)

Steel group	Toughness class	Specification	Grade	Thickness mm (in)	SMYS MPa (ksi)	Comments and recommended supplements a, b, c, d, e
III	CV2	API Spec 2W[C.9]	60	⩽ 25 (⩽ 1)	417 (60)	
		API Spec 2Y[C.10]	60	⩽ 25 (⩽ 1)	417 (60)	
		ASTM A537[C.11]	Class 2	⩽ 65 (⩽ 21/2)	417 (60)	B, C (Charpy at –40 °C (–40 °F), D
		ASTM A678[C.13]	B		417 (60)	B, C (Charpy at –40 °C (–40 ºF), D
IV	CV1	ASTM A709[C.5]	HPS70WF	⩽ 65 (⩽ 21/2)	485 (70)	B, C (per A709 supplement S84)

a ASTM A6[C.14] supplement S29 — fine austenitic grain size

b ASTM A6[C.14] supplement S74 with $P_{CE} \leqslant 0,45$ unless noted otherwise; for pressure vessel steels, see ASTM A20[C.15] supplement S20

c Charpy V-notch testing per ASTM A6[C.14] or A20[C.15] supplement S5, frequency H. For plate testing, use transverse Charpy specimens. Unless noted otherwise in steel specification, absorbed energy requirement should be as stated in Table 19.4-1.

d For joint cans add ASTM A6[C.14] supplement S8 (or A20[C.15] supplement S12), ultrasonic examination to ASTM A578[C.16] level B; and also limit $P_S \leqslant 0.006$ %.

e ASTM supplement references are from 2002 book of standards. A6[C.14] is for structural steel, A20[C.15] for pressure vessels.

Table C.3 — Correlation of steel group and toughness class for structural steel shapes to US specifications

Steel group	Toughness class	Specification	Grade	Thickness mm (in)	SMYS MPa (ksi)	Comments and recommended supplements a, b, c, d
I	NT	ASTM A36[C.1]		⩽ 50 (⩽ 2)	250 (36)	Limited availability
		ASTM A131[C.2]	A	⩽ 38 (⩽ 11/2)	235 (34)	Limited availability
	CV1	ASTM A709[C.5]	36T Zone 2		250 (36)	A, B ($P_{CE} \leqslant 0,40$), C (per A709 supplement S83)
II	NT	ASTM A572[C.6]	42	⩽ 50 (⩽ 2)	290 (42)	A [$T > 38$ mm (1,5 in)], B ($P_{CE} \leqslant 0,43$), $P_V \leqslant 0,10$
		ASTM A572[C.6]	50	⩽ 50 (⩽ 2)	345 (50)	A [$T > 38$ mm (1,5 in)], B, $P_V \leqslant 0,10$
		ASTM A992[C.17]			345 (50)	
	CV1	ASTM A709[C.5]	50T Zone 2		345 (50)	Gulf of Mexico: A, B, C (per A709 supplement S83)
		ASTM A709[C.5]	50F Zone 3		345 (50)	Cold, non-arctic: A, B, C (per A709 supplement S84)
		API Spec 2MT2[C.18]	Class B		345 (50)	Gulf of Mexico
		API Spec 2MT2[C.18]	Class A		345 (50)	Cold, non-arctic

a ASTM A6[C.14] supplement S29 – Fine austenitic grain size

b ASTM A6[C.14] supplement S74 with $P_{CE} \leqslant 0,45$ unless noted otherwise; for pressure vessel steels see ASTM A20[C.15] supplement S20

c Charpy V-notch testing per ASTM A6[C.14] or A20[C.15] supplement S5, frequency H. Test temperatures shall be as stated or per specifications cited. Unless noted otherwise in steel specification, absorbed energy requirement should be as stated in Table 19.4-1.

d ASTM supplement references are from 2002 book of standards. A6[C.14] is for structural steel, A20[C.15] for pressure vessels.

Table C.4 — Correlation of steel group and toughness class for structural steel pipe to US specifications

Steel group	Toughness class	Specification	Grade	SMYS MPa (ksi)	Comments
I	NT	API Spec 5L[C.19]	B	240 (35)	
		ASTM A53[C.20]	B	240 (35)	
		ASTM A106[C.21]	B	240 (35)	
		ASTM A135[C.22]	B	240 (35)	
		ASTM A139[C.23]	B	240 (35)	
	CV1	ASTM A106[C.21]	B	240 (35)	Normalized; Charpy at 5 °C (40 °F); recheck yield strength and tensile strength after normalizing
	CV2	ASTM A333[C.24]	6	240 (35)	
II	NT	ASTM A500[C.25]	B	290 (42)	Circular (CHS)
		ASTM A500[C.25]	B	317 (46)	Rectangular (RHS)
	CV1	API Spec 5L[C.19]	X52	358 (52)	SR-5B Charpy requirement 40 J (30 ft-lbs) at –20 °C (0 °F)

NOTE In addition to the above, API Spec 2B[C.26] structural pipe made from any of the plates listed in Table C.2 may be used.

Annex D
(informative)

Design class approach

D.1 General

Selection of the steel toughness class shall be based on a systematic classification of welded members and joints according to the structural significance and complexity of the joints. The main criteria for the determination of the appropriate design class of a component (a joint or a member) are the component's significance with respect to global integrity of the structure and the consequences of its failure. In addition, the degree of redundancy, the design uncertainties (geometrical complexity), and level of multiaxial stress of a joint will influence the design class selection. The design class approach also applies to temporary structures.

The corresponding inspection requirements are described in Annex F.

The selection of component design class shall be in compliance with Table D.1; typical design classes are shown in Figure D.1.

The typical relationship between design class and toughness class is shown in Tables D.2 and D.3. In all cases, the appropriate toughness class should be selected based on the considerations described in Clause 19. Examples of the selection of steel toughness class are shown in Table D.3. However, the designer should use best judgment in applying the principles of Table D.1 to each particular situation.

Table D.1 — Design class — Typical classification of structural components

Design class	Component complexity[a]	Consequences of failure
DC 1	High	Applicable for joints and members where failure will have substantial consequences[b] and the structure possesses limited residual strength[c]
DC 2	Low	
DC 3	High	Applicable for joints and members where failure will be without substantial consequences[b] due to residual strength [c]
DC 4	Low	
DC 5	Any	Applicable for joints and members where failure will be without substantial consequences[b]

[a] High joint complexity means joints where the geometry of connected elements and weld type leads to high restraint and to a triaxial stress pattern. E.g. typically multi-planar plated connections with full penetration welds and also unstiffened leg nodes subject to high stress concentration.

[b] "Substantial consequences" in this context means that failure of the joint or member will entail either

— danger of loss of human life,

— significant pollution, and/or

— major financial consequences.

[c] A structure may be assumed to have adequate residual strength if it meets the ALS requirements with the component under consideration damaged.

D.2 Specific steel selection

Specific material selection shall be shown in design drawings and specifications.

The strength groups and toughness classes are used to reference welding requirements, e.g. preheat and electrode selection, where these tend to follow the steel. Plates, sections and rolled tubular sections specified by the designer should conform to a recognized specification. Supplementary specifications can be required for steel used to fabricate items intended for particularly arduous duty or where recognized steel grades are used in a thickness that is above the stated limits.

Table D.4 shows a correlation of steel group and toughness class for steel plates to EN specifications. Tables D.5 and D.6 do the same for wide flange and tubular members. Steels that are not included in the above lists may be used, as long as they satisfy the provisions of Clauses 19 and 20, with particular attention to weldability (see also A.19.2.2.5). Selected materials shall meet the toughness requirements as given in Table E.1.

Table D.2 — Design class — Correlation between design class and steel toughness class

Design class	Toughness class [a]			
	CV2ZX/CV2Z [b]	CV2 [b]	CV1	NT
DC 1	X			
DC 2	X[c]	X		
DC 3	X[c]	X		
DC 4	X[c]		X	
DC 5				X

a X with no superscript denotes default toughness choice.

b For EN-steels CV2, CV2X and CV2ZX have identical requirements with respect to weldability.

c Selection where joint design requires tensile strength through the thickness of the plate.

Table D.3 — Design class — Typical minimum selection for structures

Component	Design class DC	Steel toughness class
Legs and main bracing system		
Leg nodes	1 or 2	CV2ZX or CV2
Leg strakes and cones [a, b]	2	CV1 or CV2
Lift-nodes — complex	1	CV2ZX
Lift-nodes — simple	2	CV2 or CV2Z
Nodes in vertical bracing	4	CV2
Vertical bracing	4	CV2
Bottle leg [b]	2	CV2
Horizontal bracing (and nodes)	4	CV1
Watertight diaphragms, ring stiffeners	2	CV2 (or CV1)
Other stiffening and plain mudmats	5	NT
Foundation system		
Cluster pile mudmat/yoke plate including stiffening [c]	4	CV1
Skirts and bucket foundation plates including stiffening	4	CV1
Shear plates and pile sleeves [c]	4	CV1
Pile sleeve catcher, cone and spacers [c]	5	NT
Piles [c]	4	CV1
Appurtenances and outfitting steel		
Riser guides/J-tubes and supports/Conductor support [d]	4	CV1
Sump caissons and supports [d]	5	NT
Outfitting [d]	4 or 5	NT or CV1

[a] If the structure has a foundation system at 4 or less corner legs;

— Upper part of corner legs and inner legs: design class = 4, steel toughness class = CV1,

— Lower part of corner legs: design class = 2, steel toughness class = CV1.

[b] For a space frame type structure with 6 or more legs, each supported by a foundation system: design class = 4, steel toughness class = CV1.

[c] If one or two pile(s) per leg: design class = 2, steel toughness class = CV2.

[d] Outfitting structures are normally of minor importance for structural safety and integrity. However, in certain cases the operational safety is directly influenced by the outfitting and special assessment is required in design and fabrication. A typical example is guides and supports for gas risers where footnote a applies.

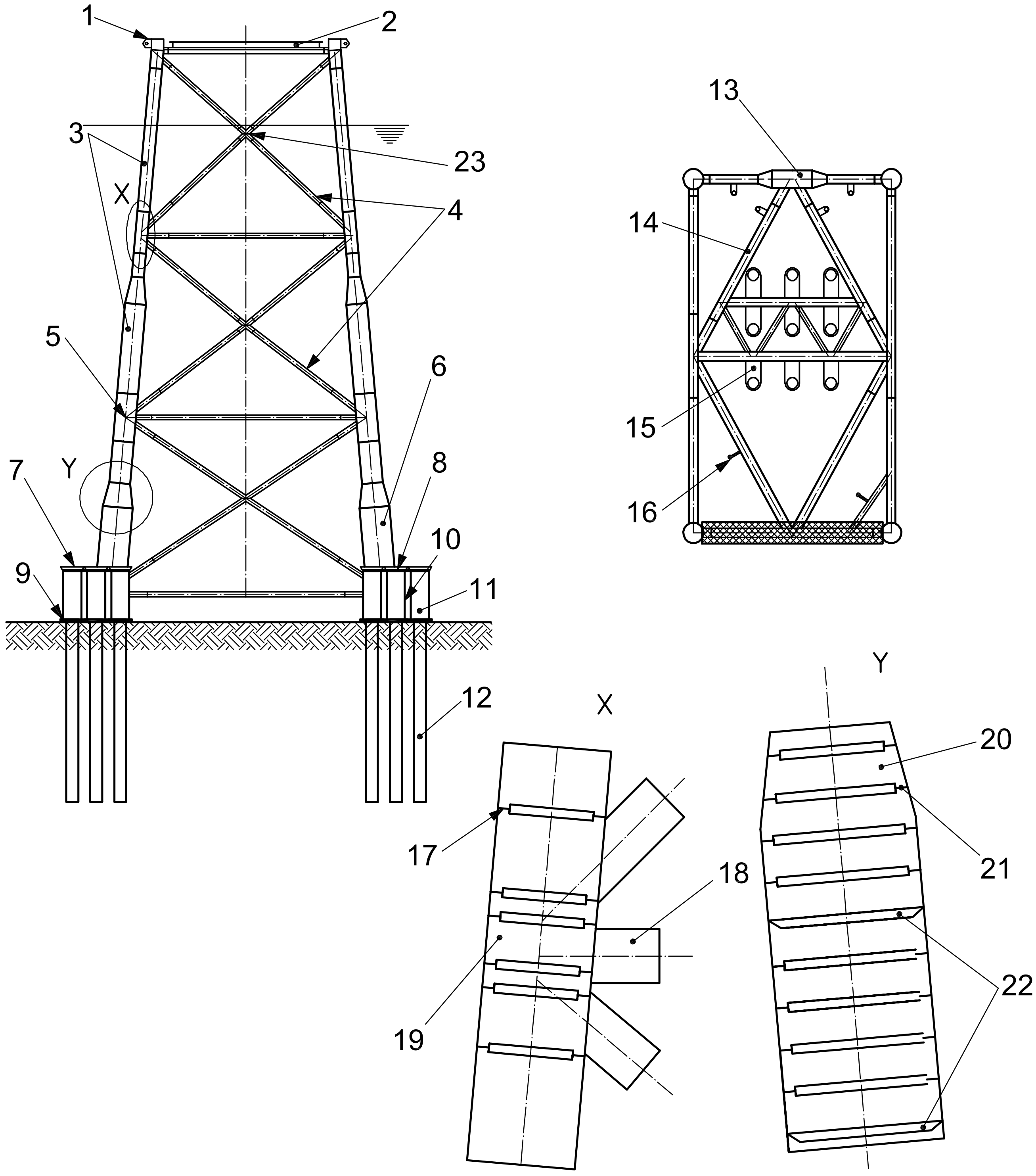


Figure D.1 — Typical design classes for four-leg steel structure with pile cluster

Key

1 lift point DC 1
2 walkways DC 5
3 main legs DC 2
4 main braces DC 4
5 main nodes, detail X DC 1, DC 2
6 bottle leg, detail Y DC 2
7 pile catcher DC 5
8 yoke plate DC 4
9 mudmat at cluster pile DC 4
10 shear plate DC 4
11 pile sleeve DC 4
12 piles DC 4
13 horizontal node DC 4
14 horizontal brace DC 4
15 conductor guide DC 4
16 riser support DC 4
17 stiffener DC 2
18 stub DC 4
19 chord DC 2
20 cone DC 2
21 stiffener DC 2
22 watertight diaphragm DC 2
23 node in vertical bracing DC 3

Figure D.1 (*continued*)

Table D.4 — Correlation of steel group and toughness class for steel plates to European specifications

Steel group	Toughness class	Specification	Grade	SMYS MPa (ksi)	Comment
I	NT	EN 10025[D.1]	S275JR/S235JRG2	255 (36)	
II	NT	EN 10025[D.1]	S355J0	355 (50)	
	CV1	EN 10025[D.1]	S355N/M	355 (50)	
		EN 10225[D.2]	S355J2G3	355 (50)	Option 5; $P_{CE} \leqslant 0,43$, $P_{S} \leqslant 0,025$, $P_{P} \leqslant 0,025$; longitudinal Charpy $\geqslant$ 40 J
		EN 10225[D.2]	S355K2G3		Option 5; $P_{CE} \leqslant 0,43$, $P_{S} \leqslant 0,025$, $P_{P} \leqslant 0,025$;
	CV2	EN 10225[D.2]	S355G9N/M	355 (50)	Options 6, 12 and 18
	CV2Z/ZX	EN 10225[D.2]	S355G10N/M	355 (50)	Options 6, 12, 13 and 18
III	CV1	EN 10025[D.1]	S420NL/ML	420 (61)	
	CV2	EN 10225[D.2]	S420G1Q/G1M	420 (61)	Options 6, 9, 12 and 18
	CV2Z/ZX	EN 10225[D.2]	S420G2Q/G2M	420 (61)	Options 6, 9, 12, 13 and 18
IV	CV2	EN 10225[D.2]	S460G1Q/G1M	460 (67)	Options 6, 9, 12 and 18
	CV2Z/ZX	EN 10225[D.2]	S460G2Q/G2M	460 (67)	Options 6, 9, 12, 13 and 18
V	CV2	NORSOK M-120[D.3]	S460G1Q/G1M modified	500 (73)	Options 6, 9, 12 and 18
	CV2Z/ZX	NORSOK M-120[D.3]	S460G2Q/G2M modified	500 (73)	Options 6, 9, 12, 13 and 18

For CV2, CV2Z and CV2ZX materials, base material information, documentation and results of weldability tests according to EN 10225[D.2] should be established prior to delivery. The documentation of base material should include a strain aging test for group V steels, typical tensile tests and weldability tests for plates within each of the following thickness ranges, relevant for the order: 25 mm to 40 mm, 40 mm to 63 mm, 63 mm to 100 mm and 100 mm to 150 mm, for both the AW and PWHT conditions. CTOD testing shall be included for thicknesses above 40 mm and shall meet the requirements of a minimum 0,20 for PWHT and 0,25 mm for AW conditions.

Table D.5 — Correlation of steel group and toughness class for steel sections to European specifications

Steel group	Toughness class	Specification	Grade	SMYS MPa (ksi)	Comment
I	NT	EN 10025[D.1]	S275JR/S235JRG2	255 (36)	
II	NT	EN 10225[D.2]	S355J0	355 (50)	
	CV1	EN 10025[D.1]	S355N/M	355 (50)	
		EN 10225[D.2]	S355J2G3/G4	355 (50)	Option 5; $P_{CE} \leqslant 0{,}43$, $P_S \leqslant 0{,}025$, $P_P \leqslant 0{,}025$; longitudinal Charpy $\geqslant 40$ J
		EN 10225[D.2]	S355K2G3/G4	355 (50)	Option 5; $P_{CE} \leqslant 0{,}43$, $P_S \leqslant 0{,}025$, $P_P \leqslant 0{,}025$;
	CV2	EN 10225[D.2]	S355G11N/M	355 (50)	Options 9 and 18
	CV2Z/ZX	EN 10225[D.2]	S355G12N/M	355 (50)	Options 9, 13, 18 and 21, class 2.1
III	CV1	EN 10025[D.1]	S420NL/ML	420 (61)	
	CV2	EN 10225[D.2]	S420G3M	420 (61)	Options 9 and 18
	CV2Z/ZX	EN 10225[D.2]	S420G4M	420 (61)	Options 9, 13, 18 and 21, class 2.1
IV	CV2	EN 10225[D.2]	S460G3M	460 (67)	Options 9 and 18
	CV2Z/ZX	EN 10225[D.2]	S460G4M	460 (67)	Options 9, 13, 18 and 21, class 2.1
V	CV2	NORSOK M-120[D.3]	S460G3M modified	500 (73)	Options 9 and 18
	CV2Z/ZX	NORSOK M-120[D.3]	S460G4M modified	500 (73)	Options 9, 13, 18 and 21, class 2.1
For CV2, CV2Z and CV2ZX materials, base material information, documentation and results of weldability tests according to EN 10225[D.2] should be established prior to delivery. The documentation of base material should include strain aging test for group V steels, typical tensile tests and weldability tests for sections within each of the following thickness ranges, relevant for the order: 25 mm to 40 mm, 40 mm to 63 mm, 63 mm to 100 mm and 100 mm to 150 mm, for both the AW and PWHT conditions. CTOD testing shall be included for thickness above 40 mm and should meet the requirements of a minimum 0,25 mm for AW condition.					

Table D.6 — Correlation of steel group and toughness class for steel tubulars to European specifications

Steel group	Toughness class	Specification	Grade	SMYS MPa (ksi)	Comment
I	NT	EN 10210[D.4]	S275J0H/S235JRH	255 (36)	Hot finished
		EN 10219[D.5]	S275J0H/S235JRH	255 (36)	Cold formed
II	NT	EN 10210[D.4]	S355J0H	355 (50)	Hot finished
		EN 10219[D.5]	S355J2H	355 (50)	Cold formed
	CV1	EN 10225[D.2]	S355G1N	355 (50)	Hot finished
		EN 10210[D.4]	S355NH	355 (50)	Hot finished, option 1.4, $P_{CE} \leqslant 0{,}43$
		EN 10219[D.5]	S355MLH	355 (50)	Cold formed, option 1.4, $P_S \leqslant 0{,}015$, $P_P \leqslant 0{,}025$
	CV2	EN 10225[D.2]	S355G14Q/N	355 (50)	Options 6, 7 and 18; $P_C \leqslant 0{,}16$
	CV2Z/ZX	EN 10225[D.2]	S355G15Q/N	355 (50)	Options 6, 7, 13, 18 and 22; $P_C \leqslant 0{,}16$
III	CV1	EN 10219[D.5]	S420MLH	420 (61)	Cold formed, $P_{CE} \leqslant 0{,}39$, $P_S \leqslant 0{,}015$, $P_P \leqslant 0{,}025$; Charpy $\leqslant$ 50 J
	CV2	EN 10225[D.2]	S420G6Q modified	420 (61)	Options 6 and 18
	CV2Z/ZX	EN 10225[D.2]	S420G6Q modified	420 (61)	Options 6, 13, 18 and 22
IV	CV2	EN 10225[D.2]	S460G6Q modified	460 (67)	Options 6 and 18
	CV2Z/ZX	EN 10225[D.2]	S460G6Q modified	460 (67)	Options 6, 13, 18 and 22
V	CV2	NORSOK M-120[D.3]	S460G6Q modified	500 (73)	Options 6, 12 and 18
	CV2Z/ZX	NORSOK M-120[D.3]	S460G6Q modified	500 (73)	Options 6, 12, 13, 18 and 22
For CV2, CV2Z and CV2ZX materials, base material information, documentation and results of weldability tests according to EN 10225[D.2] should be established prior to delivery. The documentation of base material should include a strain aging test for group V steels, typical tensile tests and weldability tests for tubulars within each of the following thickness ranges that are relevant for the order: 25 mm to 40 mm, 40 mm to 63 mm, 63 mm to 100 mm and 100 mm to 150 mm, for both the AW and PWHT conditions. CTOD testing shall be included for thickness above 40 mm and should meet the requirements of a minimum 0,25 mm for AW condition.					

D.3 Welding and non-destructive inspection categories

The designer shall select an inspection category for each component from Tables D.7 to D.9. This information shall be provided to the fabrication contractor. Inspection categories for non-visual inspection for the DC method are described in Tables D.7 and D.8. For high fatigue utilization (i.e. calculated fatigue life less than 3 times the required fatigue life), Table D.8 applies. For fatigue lives more than three times the required life, Table D.7 applies. Typical minimum selections of toughness and non-destructive weld inspection categories can be found in Table D.9. The extent of inspection by type of weld in each category is described in Table F.2.

Table D.7 — Design class — Determination of inspection category for details with low fatigue utilization (connections with calculated fatigue life longer than 3× required fatigue life)

Design class	Type and level of stress and direction in relation to welded joint	Inspection category [a]
DC 1 and DC 2	Welds subjected to high tensile stresses transverse to the weld [b]	A
	Welds with moderate tensile stresses transverse to the weld and/or high shear stresses [c]	B [d]
	Welds with low tensile stresses transverse to the weld and/or moderate shear stress [e]	C [f]
DC 3 and DC 4	Welds subjected to high tensile stresses transverse to the weld [b]	B [d]
	Welds with moderate tensile stresses transverse to the weld and/or high shear stresses [c]	C [f]
	Welds with low tensile stresses transverse to the weld and/or moderate shear stress [e]	D [g]
DC 5	All load bearing connections	D
	Non load bearing connections	E

[a] It is recommended that areas of the welds where stress concentrations occur be marked as mandatory inspection areas for B, C and D categories, as applicable.

[b] High tensile stresses mean ULS tensile stresses in excess of 0,85 of design stress.

[c] Moderate tensile stresses mean ULS tensile stresses between 0,6 and 0,85 of design stress.

[d] Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category A.

[e] Low tensile stresses mean ULS tensile stresses less than 0,6 of design stress.

[f] Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category B.

[g] Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category C.

Table D.8 — Design class — Determination of inspection category for details with high fatigue utilization (connections with calculated fatigue life shorter than 3× required fatigue life)

Design class	Direction of dominating principal stress	Inspection category [a]
DC 1 and DC 2	Welds with the direction of the dominating time-varying principal stress transverse to the weld (between 45° and 135°)	A [b]
	Welds with the direction of the dominating time-varying principal stress in the direction of the weld (between −45° and 45°)	B [c]
DC 3 and DC 4	Welds with the direction of the dominating time-varying principal stress transverse to the weld (between 45° and 135°)	B [c]
	Welds with the direction of the dominating time-varying principal stress in the direction of the weld (between −45° and 45°)	C [d]
DC 5	Welds with the direction of the dominating time-varying principal stress transverse to the weld (between 45° and 135°)	D
	Welds with the direction of the dominating time-varying principal stress in the direction of the weld (between −45° and 45°)	E

[a] For joints in inspection categories B, C or D, the regions of geometrical stress GS (regions with the highest stress range) at welds or areas of welds of special concern shall be addressed with individual notations as mandatory for selected NDT methods.

[b] Butt welds with high fatigue utilization and stress concentration factor (SCF) less than 1,3 need stricter NDT acceptance criteria. Such criteria should be developed in each case.

[c] Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category A.

[d] Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category B.

Table D.9 — Design class — Typical minimum selection for structures

Joint/ component	Design class DC	Toughness class	Inspection category [a]	Comment
Legs and main bracing system				
Leg nodes and cones/All welds	1 or 2	CV2 or CV2ZX	A or B	[c, d]
Leg nodes and cones/Longitudinal welds	2	CV2 or CV2ZX	B	[c, d]
Leg strakes/Circumferential welds	2	CV2	A or B	[c, d]
Leg strakes/Longitudinal welds	2	CV2	B	[c, d]
Lift-nodes - complex	1	CV2ZX	A or B	
Lift-nodes - simple	2	(CV2ZX)/CV2	A or B	
Nodes in vertical bracing/Circumferential welds	4	CV2	B	
Nodes in vertical bracing/Longitudinal welds	4	CV2	C	
Vertical bracing/Circumferential welds	4	CV2	B	
Vertical bracing/Longitudinal welds	4	CV2	C	
Bottle leg/Circumferential welds	2	CV2	A or B	
Bottle leg/Longitudinal welds	2	CV2	B	
Horizontal bracing/All welds	4	CV1	B or C	
Nodes horizontal bracings/All welds	4	CV1	B or C	
Watertight diaphragms/All welds	2	CV2 or CV1	A or B	
Ring stiffeners, main nodes/All welds	2	CV2 or CV1	A	
Ring stiffeners bottle leg/All welds	2	CV2 or CV1	A	
Other stiffening/All welds	5	CV1	D or E	
Foundation system				
Mudmat and yoke plate incl. stiffening/All welds	4	CV1	C	[e]
Skirts and bucket foundation plates including stiffening/All welds	4	CV1	C	
Shear plates/All welds	4	CV1	C	[e]
Pile sleeves/All welds	4	CV1	C	[e]
Pile sleeve catcher, cone and spacers	5	NT	D or E	
Piles, top part/Butt welds	4	CV1	A	[e]
Piles, top part/Longitudinal welds	4	CV1	A	[e]
Piles, remaining/All welds	4	CV1	A	[e]
Appurtenances and outfitting steel				
Riser guides/All welds	4	CV1	B or C	[b]
J-tubes and supports/All welds	4	CV1		
Conductor support/All welds	4	CV1		
Caissons and support/All welds	5	NT		
Outfitting/Butt welds	4 or 5	CV1 or NT	D or E	[b]
Outfitting/Part-Pen. and Fillets	4 or 5	CV1 or NT	D or E	[b]

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

[b] Outfitting structures are normally of minor importance for structural safety and integrity. However, in certain cases the operational safety is directly influenced by the outfitting and special assessment is required in design and fabrication. A typical example is guides and supports for gas risers.

[c] If multi-legged space frame structures with corner legs supported by foundation systems:

— upper part of corner legs and inner legs, minimum selection is: DC 4, CV1, B or C;

— lower part of corner legs: DC 2, CV2, A or B.

[d] If multi-legged space frame structures with each leg supported by a foundation system: DC 4, CV1, B or C.

[e] If one or two pile(s) per leg: DC2 and CV2.

Annex E
(informative)

Welding and weld inspection requirements — Material category approach

E.1 General

This annex contains the weld toughness and non-destructive testing requirements for the fabrication, welding, and inspection of fixed steel offshore structures which have been designed using the MC methodology.

It is anticipated that the material category (MC number), steel group (strength level) and toughness class will have been determined by specific steel selections given in the design drawings and specifications.

The weld toughness requirements follow a hierarchy that parallels the toughness class of the steels to be joined, and which has been previously defined (see Clause 19) in consideration of criticality, strength level, thickness, and other aspects of the corresponding structural component.

The NDT inspection requirements use a hierarchy which follows the structural component type directly.

E.2 Weld toughness

E.2.1 Weld metal toughness

Table E.1 gives the minimum weld metal toughness requirements. These shall be demonstrated by testing during welding procedure qualification. An exception is provided for low-criticality applications, where as-classified welding consumable toughness may be relied upon.

E.2.2 HAZ toughness

Table E.2 gives the minimum HAZ toughness requirements. For critical applications as indicated, these shall be demonstrated by tests during welding procedure qualification. CTOD testing of the HAZ is required only for CV2X and CV2ZX steels, only in MC1 structures, only for thicknesses over 50 mm, and only where not covered by a manufacturer’s steel pre-qualification.

Table E.1 — Material category — Minimum weld metal toughness requirements

Steel group	Toughness class	MC1		MC2		MC3	
		Energy J (ft-lbs)	Temperature below LAST °C (°F)	Energy J (ft-lbs)	Temperature below LAST °C (°F)	Energy J (ft-lbs)	Temperature below LAST °C (°F)
I	NT	27 (20)	10 (18) [a]	27 (20)	10 (18) [a]	27 (20)	10 (18) [a]
	CV1	27 (20)	10 (18) [a]	27 (20)	10 (18) [a]	N/A	N/A
	CV2	27 (20)	20 (36)	27 (20)	20 (36)	N/A	N/A
II	NT	27 (20)	20 (36) [a]	27 (20)	10 (18) [a]	27 (20)	10 (18) [a]
	CV1	27 (20)	10 (18)	27 (20)	10 (18)	27 (20)	10 (18)
	CV2	34 (25)	20 (36)	34 (25)	20 (36)	34 (25)	20 (36)
	CV2Z	34 (25)	20 (36)	34 (25)	30 (54)	34 (25)	30 (54)
	CV2ZX	34 (25)	30 (54)	34 (25)	30 (54)	N/A	N/A
III	CV2	40 (30)	20 (36)	40 (30)	20 (36)	N/A	N/A
	CV2Z	40 (30)	30 (54)	40 (30)	30 (54)	N/A	N/A
	CV2Z	40 (30)	30 (54)	N/A	N/A	N/A	N/A
	CV2ZX	40 (30)	30 (54)	40 (30)	30 (54)	N/A	N/A
IV	CV2ZX	40 (30)	10 (18)	N/A	N/A	N/A	N/A

Steel group V not yet included: the toughness requirement shall be specified by the user.

N/A indicates that this category is not normally applicable. Where such steels are present, use the nearest toughness requirements in the same row of the table.

[a] CVN test shall not be required if the electrode specification has a minimum CVN value that is greater than or equal to 27 J (20 ft-lbs) satisfying the test temperature or lower.

Table E.2 — Material category – minimum heat affected zone toughness requirements

Steel group	Toughness class	MC1		MC2		MC3	
		Energy J (ft-lbs)	Temperature below LAST °C (°F)	Energy J (ft-lbs)	Temperature below LAST °C (°F)	Energy J (ft-lbs)	Temperature below LAST °C (°F)
I	NT	n/a	N/A	N/A	N/A	N/A	N/A
	CV1	20 (15)	0 (0)	20 (15)	0 (0)	N/A	N/A
	CV2	20 (15)	10 (18)	N/A	N/A	N/A	N/A
II	NT	N/A	N/A	N/A	N/A	N/A	N/A
	CV1	27 (20)	0 (0)	27 (20)	0 (0)	20 (15)	0 (0)
	CV2	34 (25)	10 (18)	34 (25)	10 (18)	34 (25)	0 (0)
	CV2Z	34 (25)	10 (18)	34 (25)	10 (18)	34 (25)	0 (0)
	CV2ZX	34 (25)	20 (36)	34 (25)	20 (36)	N/A	N/A
III	CV2	40 (30)	10 (18)	40 (30)	10 (18)	N/A	N/A
	CV2Z	40 (30)	10 (18)	40 (30)	10 (18)	N/A	N/A
	CV2ZX	40 (30)	20 (36)	40 (30)	20 (36)	N/A	N/A
IV	CV1	40 (30)	0 (0)	N/A	N/A	N/A	N/A

CTOD testing of the HAZ shall not be required if the steel has been pre-production tested in the HAZ and the production WPS heat input range is within the pre-production test range.

N/A indicates that this category is not required.

E.3 Inspection

See Table E.3 for the extent of non-destructive testing (NDT) requirements.

Proportional inspection rates are further described in A.20.3.

Personnel requirements and reject criteria for NDT, other required inspections (e.g. visual and dimensional checks), and document requirements are described in

a) the selected generic standard for welding, fabrication, and inspection,

b) Annex G fabrication tolerances,

c) the project quality control plan,

d) the owner's stipulated quality assurance requirements, and/or

e) the contractor's accredited quality management system (see 21.2).

Table E.3 — Material category approach — Minimum extent of weld inspection

Component	Material category and inspection method					
	MC1		MC2		MC3	
	UT/RT [a]	MPI	UT/RT [a]	MPI	UT/RT [b]	MPI
Fabricated pipe and braces						
Primary circumferential welds	100 %	100 % [c]	100 %		100 % [c]	
Secondary circumferential welds	10 %	10 %	10 %		10 %	
Long seams in node cans	100 %	100 % [c]	100 %		10 %	
All other long seams	100 % [b, c]		10 % [d]		10 % [d]	
Tubular joints						
Primary tubular joints (including secondary to primary)	100 %	100 %	100 %	100 % [c]	100 %	
Secondary tubular joint	100 % [b, c]	100 %	10 % [b]	100 % [c]		100 % [c]
Deck						
Trusses – full penetration welds	100 %	100 % [c]	100 % [c]		10 %	
Full penetration welds – other	100 % [c]	100 % [c]	10 %			
Partial penetration/ fillet welds		100 % [c]		10 %		
Plate girders – web to flange						
At major brace connections – full pen.	100 %	100 % [c]	100 % [c]	10 %	10 %	
Full penetration – other locations	100 %	100 % [c]		10 %		
Partial penetration/ fillet welds		100 % [c]		10 %		
Piling, conductors						
Circumferential welds	100 %	100 %	100 %		100 %	
Pile to shim connection		100 %		100 % [f]		100 % [f]
Pad eyes/lifting aids/heavy lift frames						
Full penetration welds	100 %	100 %	100 %	100 %	100 %	100 %
Partial penetration/ fillet welds		100 % [e]		100 % [e]		100 % [e]
Secondary steelwork						
Boat bumpers, landings	100 % [f]	10 %	100 % [f]			
Stairways, walkways, etc.		10 %				
Launch/launch legs						
Full penetration welds	100 %	100 % [c]	100 %		100 %	
Partial penetration/ fillet welds		100 %		100 %		100 %
Stiffeners, rings						
Full penetration welds	100 %	100 % [c]	10 %	100 % [c]		10 %
Partial penetration/ fillet welds		100 % [c]		10 %		10 %

Table E.3 (*continued*)

Component	Material category and inspection method					
	MC1		MC2		MC3	
	UT/RT [a]	MPI	UT/RT [a]	MPI	UT/RT [b]	MPI
Other items						
Crane pedestal	100 %	100 %	100 %	100 %	100 %	100 %
Pipeline riser supports	100 %	100 %	100 %	100 %	100 %	100 %
Vent/flare tower to structure	100 %	100 %	100 %	100 %	100 %	100 %
Overhead crane beam	100 %	100 %	100 %	100 %	10 %	10 %

[a] Use RT for $t < 10$ mm ($t < 3/8$ in), otherwise use UT or RT.

[b] As an exception to footnote a above, when thickness $t < 10$ mm ($t < 3/8$ in), MPI shall be performed in lieu of UT/RT for noted items.

[c] This may be reduced to 10 % when a repair rate below 2 % is maintained for UT/RT inspections or 0,2 % for MPI.

[d] To include both ends.

[e] Root pass and final weld to be inspected.

[f] UT/RT applies only when welding to primary steel.

Annex F
(informative)

Welding and weld inspection requirements — Design class approach

F.1 General

This annex contains welding and inspection requirements for the DC methodology.

F.2 Toughness of weld and heat affected zone (HAZ)

F.2.1 General

Table F.1 gives the weld metal and HAZ Charpy toughness requirements, which are to be demonstrated during WPQ, depending on material strength group and thickness.

For NT steels, HAZ testing is not required. For CV1 and higher toughness classes, there is no requirement for the HAZ toughness to be higher than that for the base metal. Stress relieving heat treatment, where applied, shall be represented in the WPQ test specimens. Charpy testing and related essential variables are described in 20.2.2.4.

F.2.2 CTOD testing

CTOD testing for CV2ZX and CV2X steels, and where designated for other steels in Annex D (e.g. in lieu of stress relieving for certain thicknesses), is described in 20.2.2.5 and Annex B. CTOD can also apply for fitness-for-purpose evaluations.

F.2.3 PWHT alternative to CTOD testing

Where the drawings give no indication, all welds with a minimum design throat thickness exceeding 40 mm (1 9/16 in) on nodes and 50 mm (2 in) elsewhere shall be post weld heat treated, or subjected to a full fracture mechanics assessment of the welds under consideration. In such cases the designer will advise the CTOD acceptance levels.

F.3 Extent of NDT for structural welds

Table F.2 describes the minimum extent of non-destructive testing for structural welds when using the design class approach.

Table F.1 — Design class — Minimum weld metal and HAZ toughness requirements (test temperatures and energy requirements)

Material thickness mm	Steel toughness class				
	NT	CV1		CV2/CV2Z/CV2ZX	
	SMYS ⩽ 400 MPa Group I and II	SMYS ⩽ 400 MPa Group I and II	SMYS > 400 MPa Group III	SMYS ⩽ 400 MPa Group I and II	SMYS > 400 MPa Group III, IV and V[b]
$t \leqslant 12$ ($t \leqslant 0{,}5$ in)	LAST +10 °C (LAST +18 °F)	LAST +10 °C (LAST +18 °F)	LAST +10 °C (LAST +18 °F)	LAST +10 °C (LAST +18 °F)	LAST -10 °C (LAST -18 °F)
$12 < t < 25$ ($0{,}5$ in $< t < 1{,}0$ in)	LAST +10 °C (LAST +18 ºF)	LAST +10 °C (LAST +18 °F)	LAST -10 °C (LAST -18 °F)	LAST -10 °C (LAST -18 °F)	LAST -30 °C (LAST -54 °F)
$25 \leqslant t < 50$ ($1{,}0$ in $\leqslant t < 2{,}0$ in)	LAST -10 °C (LAST -18 °F)	LAST -10 °C (LAST -18 °F)	LAST -30 °C (LAST -54 °F)	LAST -30 °C (LAST -54 °F)	LAST -30 °C (LAST -54 °F)
$t \geqslant 50$ ($t \geqslant 2{,}0$ in)	Combination not allowed	LAST -30 °C (LAST -54 °F)	LAST -30 °C (LAST -54 °F)	LAST -30 °C (LAST -54 °F)	LAST -30 °C (LAST -54 °F)
Energy requirement [a]	27 J (20 ft-lbs)	27 J (20 ft-lbs)	40 J (30 ft-lbs)	34 J (25 ft-lbs)	40 J (30 ft-lbs)

[a] No individual specimen test value shall be less than 70 % of the minimum average value described in this table.

[b] Higher energy should be considered for group IV and V used in DC1 applications.

Table F.2 — Design class – minimum extent of non-destructive testing for structural welds

Inspection category	Type of connection	Visual examination	Extent of testing		
			Radiographic testing (RT)	Ultrasonic testing (UT)	Magnetic particle testing (MPI)
A [a]	Butt weld	100 %	10	100 %	100 %
	T-connection [d]	100 %	—	100 %	100 %
	Fillet/partial	100 %	—	—	100 %
B [a, b]	Butt weld	100 %	Spot [c]	50 % [e, f]	100 % [e, f]
	T-connection [d]	100 %	—	50 % [e, f]	100 % [e, f]
	Fillet/partial	100 %	—	—	100 % [e, f]
C [b]	Butt weld	100 %	—	20 % [e, f]	20 % [e, f]
	T-connection [d]	100 %	—	20 % [e, f]	20 % [e, f]
	Fillet/partial	100 %	—	—	20 % [e, f]
D	All welds	100 %	—	—	Spot [c]
E	All welds	100 %	—	—	—

[a] In addition to the requirements in this table, the following shall apply for category A and B welds.

1) One RT film at each end and one in the middle for longitudinal welds of tubulars (including tubulars for nodes and stubs).
2) Where radiographic testing is required, intersection welds, and those locations where presence of defects is deemed to be most harmful, shall be tested.
3) Ultrasonic and radiographic testing shall not overlap, except when 100 % UT is specified. However, ambiguous imperfections revealed by UT shall in addition be tested by RT.
4) Ultrasonic testing is normally not applicable for thickness less than 10 mm. For such thickness, UT may be replaced with RT. In general, RT should be considered if UT is not possible. Radiographic testing is normally not applicable for thickness above 40 mm.
5) MPI shall be performed on both external and internal surfaces as accessible.

[b] During initial fabrication, the extent of UT and MPI of inspection category B and C welds shall be intensified, normally to twice the level given in this table. This extent shall be maintained for a weld and test length sufficient to conclude that the weld repair percentage is at a reasonable level. The increased initial testing may be accounted for in the overall extent provided the initial testing confirms consistent good workmanship.

[c] Spot means approximately 5 %.

[d] Includes complete joint penetration welds in tubular T ,Y, and K-connections and other applications.

[e] The extent may be reduced to 50 % of the specified extent, based on experience and documented records with similar joints, provided the defect rate for UT/RT is $<$ 2,0 % and for MPI is $<$ 0,2 % during the last 100 m of weld. The last 100 m shall be continuously updated every week. If the defect rate exceeds the limits given above, the normal extent of NDT shall apply again. A possible reduction in the extent of NDT shall be considered separately for each welding method and each production area.

[f] The extent of NDT shall be increased for inspection categories B, C and D if repeated occurrence of planar defects are revealed or if the weekly defect rate for any NDT method, including all types of defects, exceed the rates shown in A.20.3-1. The increase in extent of NDT shall be in accordance with A.20.3-1.

Annex G
(normative)

Fabrication tolerances

G.1 Measurements

Tolerances shall be based on theoretical setting out points and centre lines of the structure referenced to permanent approved datum points (e.g. coordinated survey stations) and corrected to a temperature of +20 °C (68 °F). Compensation shall be made for significant deflection differences between temporary support and final conditions.

Fabrication and yard assembly supports shall be set to within ± 5 mm (1/4 in) of the appropriate position shown on the approved cutting out drawings. Where no such drawings exist, fabrication shall be carried out from a level plane to within ± 5 mm (1/4 in).

G.2 Launch rails

The dimensional tolerance of launch rail centre lines shall be within ± 20 mm (3/4 in) of the theoretical position and shall also be within ± 6 mm (1/4 in) of its reference elevation. The variation in elevation between any two points on a launch rail shall not exceed 3 mm (1/8 in) within any 3 m (10 ft).

G.3 Global horizontal tolerances

The global tolerances for leg spacing at plan bracing levels are as shown in Figure G.1 and detailed below:

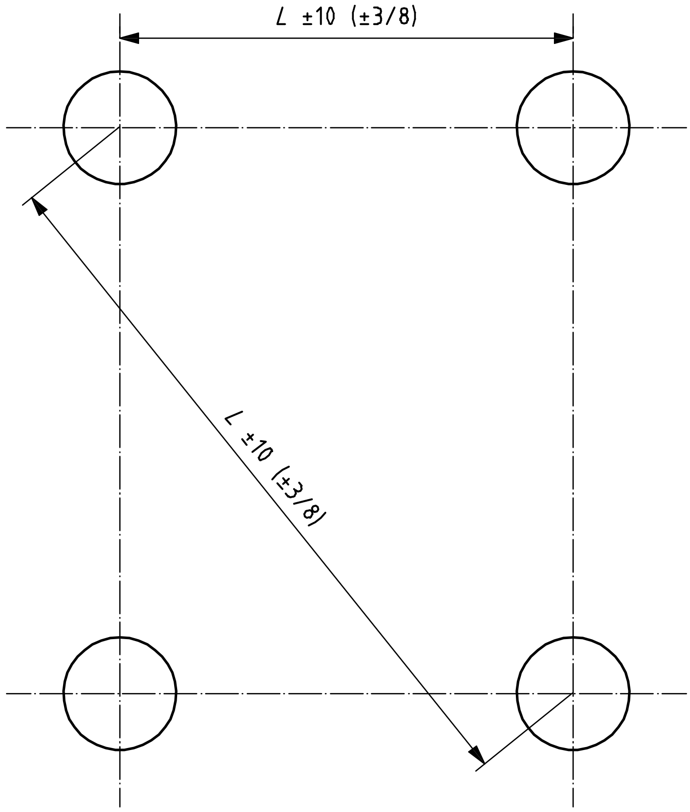
a) the horizontal centre to centre distance between adjacent legs at the top of a structure where a deck or other structure is to be placed (stab-in nodes) shall be within 10 mm (3/8 in) of the design values;

b) the horizontal centre-to-centre distance between legs at other locations shall be within 20 mm (3/4 in) of the design values;

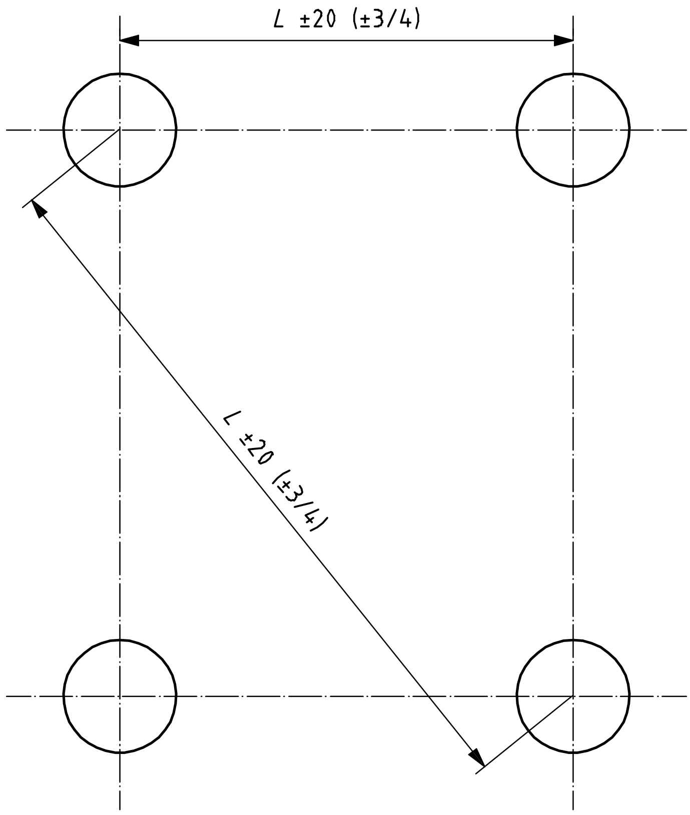
c) the horizontal centre-to-centre diagonal distances between legs at the top of a structure where a deck or other structure is to be placed (stab-in nodes) shall be within 10 mm (3/8 in) of the design values;

d) the horizontal centre-to-centre diagonal distance between legs at other locations shall be within 20 mm (3/4 in) of the design values.

Dimensions in millimetres (inches)



a) Top elevation for deck stab-in



b) Other elevations

Figure G.1 — Global horizontal tolerances

G.4 Global vertical tolerances

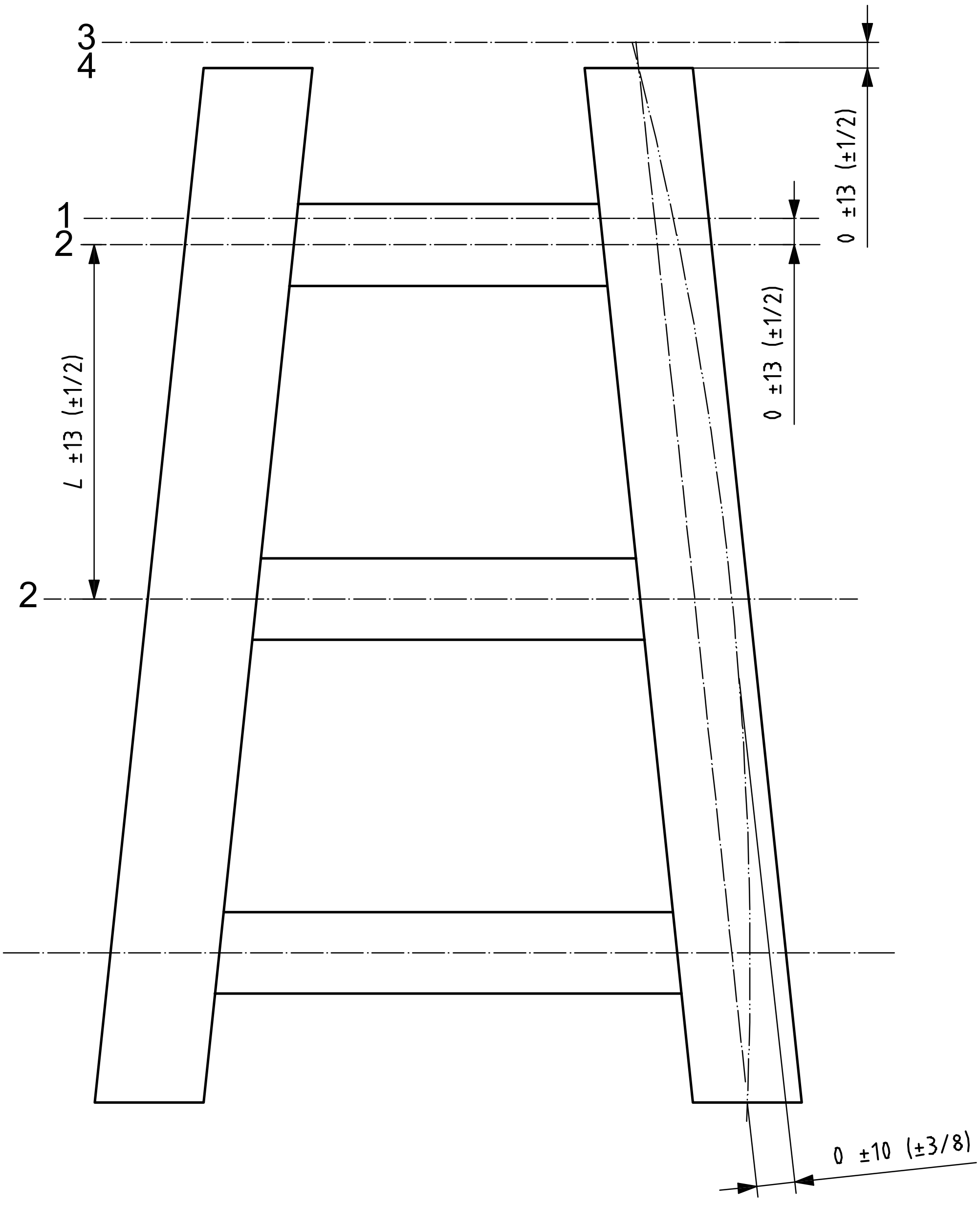
The global tolerances for vertical levels of plan bracing are as shown in Figure G.2 and are detailed as follows:

a) the elevation of plan bracing levels shall be within 13 mm (1/2 in) of the design values;

b) the vertical level of braces within a horizontal plane shall be within 13 mm (1/2 in) of the design values;

c) the vertical distance between plan bracing elevations shall be within 13 mm (1/2 in) of the design values.

Dimensions in millimetres (inches)



Key

1 theoretical centreline
2 actual centreline
3 theoretical top of legs
4 actual top of legs

Figure G.2 — Global vertical tolerances

G.5 Roundness of tubular members

For tubular members with thicknesses of 50 mm (2 in) or less, the difference between the major and the minor outside diameters (the out-of-roundness) at any point of the tubular member shall not exceed the smaller of either 1 % of the diameter or 6 mm (1/4 in).

For tubular members with thicknesses of more than 50 mm (2 in), the difference between the major and the minor outside diameters at any point of the tubular member shall not exceed 12,5 % of the wall thickness.

For tubular members with nominal outside diameters greater than or equal to 1 200 mm (50 in) and with wall thicknesses less than or equal to 100 mm (4 in), the maximum difference between the major and minor actual outside diameters at any section of the tubular shall be 13 mm (1/2 in), provided the actual circumference is within 6 mm (1/4 in) of the nominal circumference.

G.6 Circumference of tubular members

The difference between the actual and nominal outside circumferences at any point of a tubular member shall not exceed the lesser of either 1 % of the nominal circumference or 13 mm (1/2 in).

G.7 Straightness and circumferential weld locations of tubular members

The tolerances for the straightness of tubular members are as shown in Figures G.3 and G.4, and as follows:

a) lengths equal to or less than 12 m shall be straight to within 10 mm (3/8 in);

b) lengths exceeding 12 m shall be straight to within 13 mm (1/2 in).

Additionally, the tolerances for the straightness of tubular members when assembled into the structure are as follows:

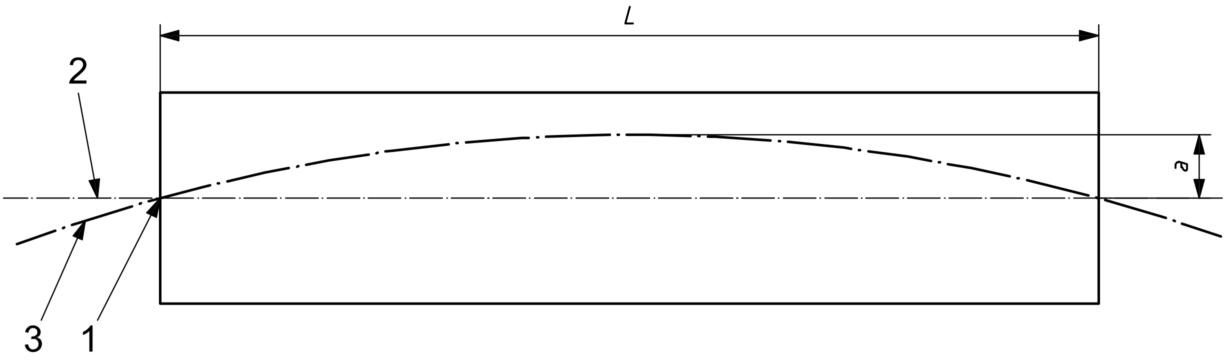
a) brace members shall be straight to within 0,12 % of the nominal length measured between work points;

b) chord members shall be straight to within 0,10 % of the nominal length measured between the work points of major framing members.

The tolerances for the location of cans with a different wall thickness to the member, and other wall thickness transitions within a tubular member, are as shown in Figure G.5 and as follows:

a) for joint cans, within 25 mm (1 in) of the design value;

b) for other changes of wall thickness, within 50 mm (2 in) of the design value.



For $L \leqslant$ 12 m (40 ft), $a \leqslant$ 10 mm (0,375 in)

For $L >$ 12 m (40 ft), $a \leqslant$ 13 mm (0,5 in)

Key

1 datum: centreline at member end

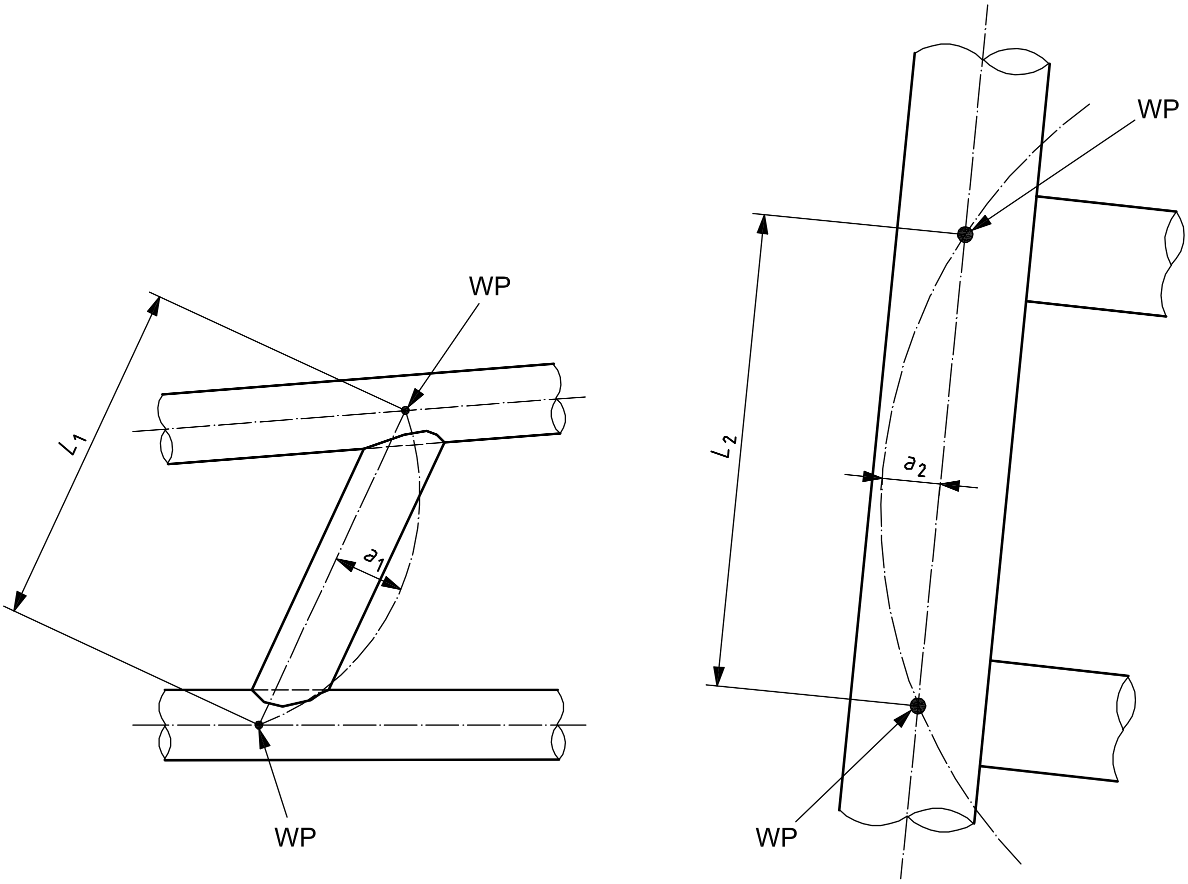
2 theoretical centreline

3 actual centreline

L total length of fabricated tubular prior to structural assembly. L may consist of one or more tubular sections welded together

a maximum actual centreline deviation from the theoretical centreline, measured normal to the theoretical centreline and anywhere within L in any plane

Figure G.3 — Tolerance on straightness of tubular members



a) Brace member

b) Chord member

$a_1 \leqslant 0{,}001\,2\,L_1$

$a_2 \leqslant 0{,}001\,0\,L_2$

Key

WP work point

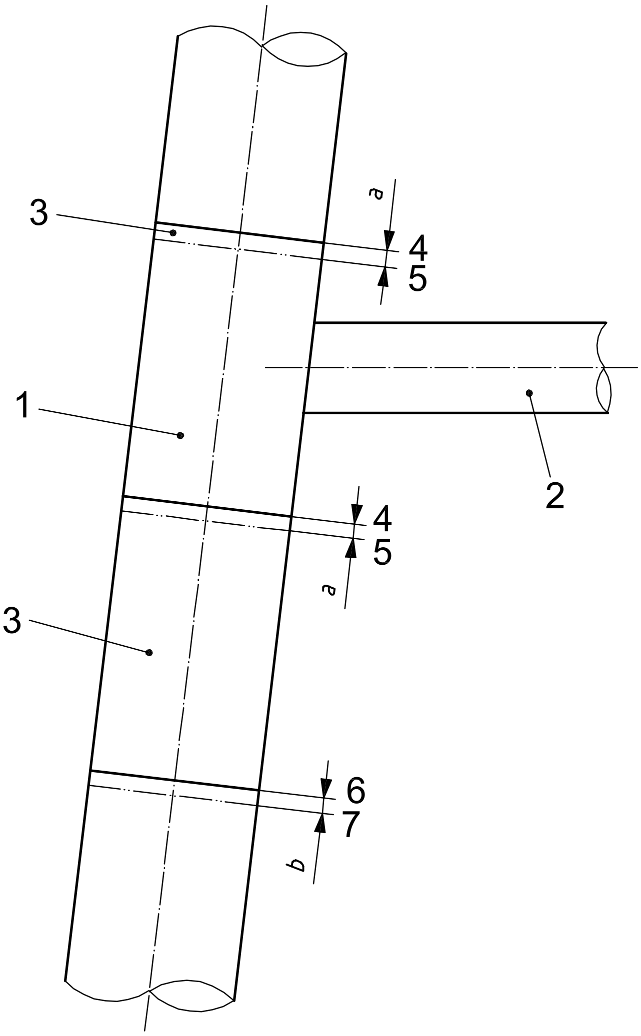
L_1 nominal length of brace member between work points

L_2 nominal length of chord member between work points of major framing members

a_1 maximum deviation at brace centreline from theoretical centreline

a_2 maximum deviation of chord centreline from theoretical centreline established between nearest framing member working points

Figure G.4 — Tolerances on straightness of tubular members assembled into structure



$a \leqslant$ 25 mm (1,0 in)

$b \leqslant$ 50 mm (2,0 in)

Key

1 joint
2 brace
3 member
4 theoretical location of thickness transition between joint and member
5 actual location of thickness transition between joint and member
6 theoretical location of thickness transition within member
7 actual location of thickness transition within member

a distance between design and as-built location thickness change between joint can and member
b distance between design and as-built location thickness change within member

Figure G.5 — Tolerances on thickness change locations

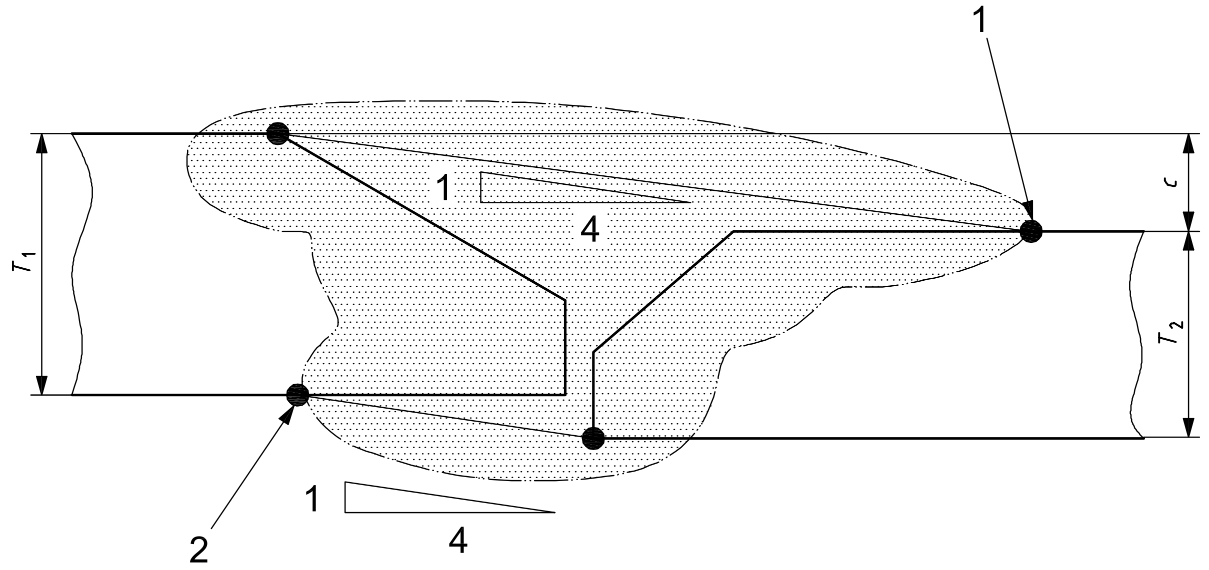
G.8 Joint mismatch for tubular members

The tolerances for the mismatch at circumferential welds and longitudinal seams in tubular members are as shown in Figure G.6 and detailed below:

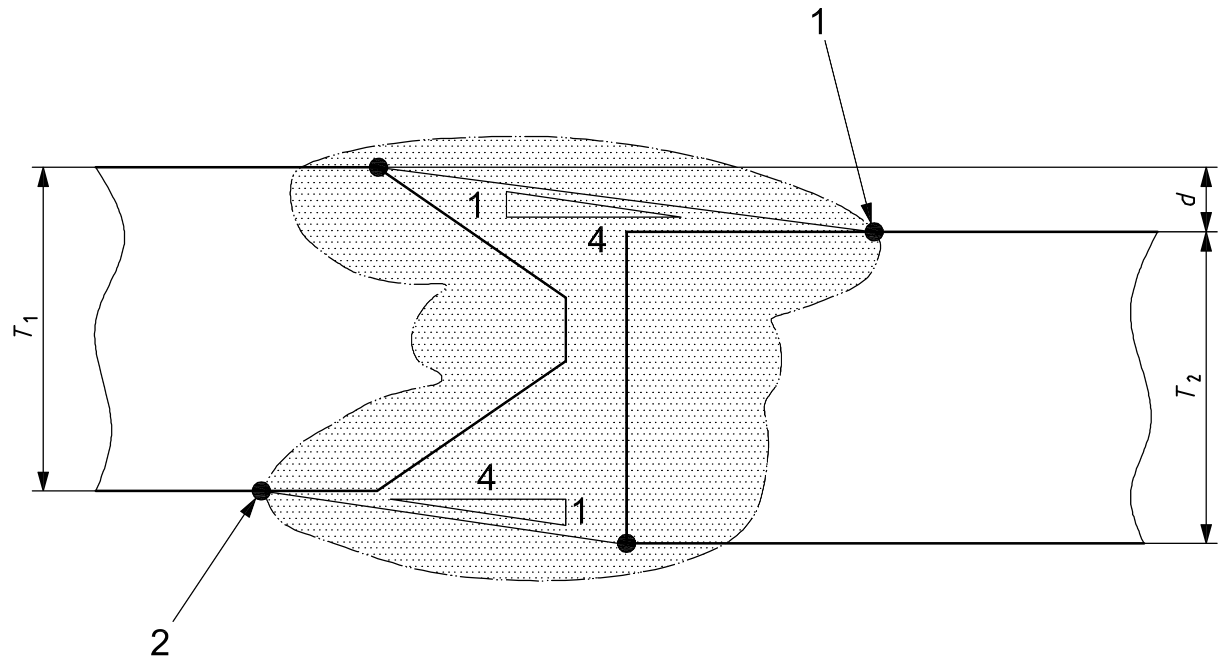
a) for double-sided welds, the lesser of 10 % of the thicker member thickness, or 6 mm (1/4 in);

b) for single-sided welds, the lesser of 10 % of the thicker member thickness, or 3 mm (1/8 in).

Where these tolerances are exceeded, the transitions shall be made by weld profiling to a maximum slope of 1:4, see Figure G.6.



a) single-sided weld



b) double-sided weld

Key

1, 2 points established by the transition weld toe that can be connected by a theoretical line to the prepared groove high point (on top surface) and low point (on bottom surface)

c less than the smaller value of 1) 0,10 T_1, 2) 0,10 T_2, 3) 3 mm (1/8 in)

d less than the smaller value of 1) 0,10 T_1, 2) 0,10 T_2, 3) 6 mm (1/4 in)

NOTE Grooves shown are for illustration only.

Figure G.6 — Joint mismatch tolerances

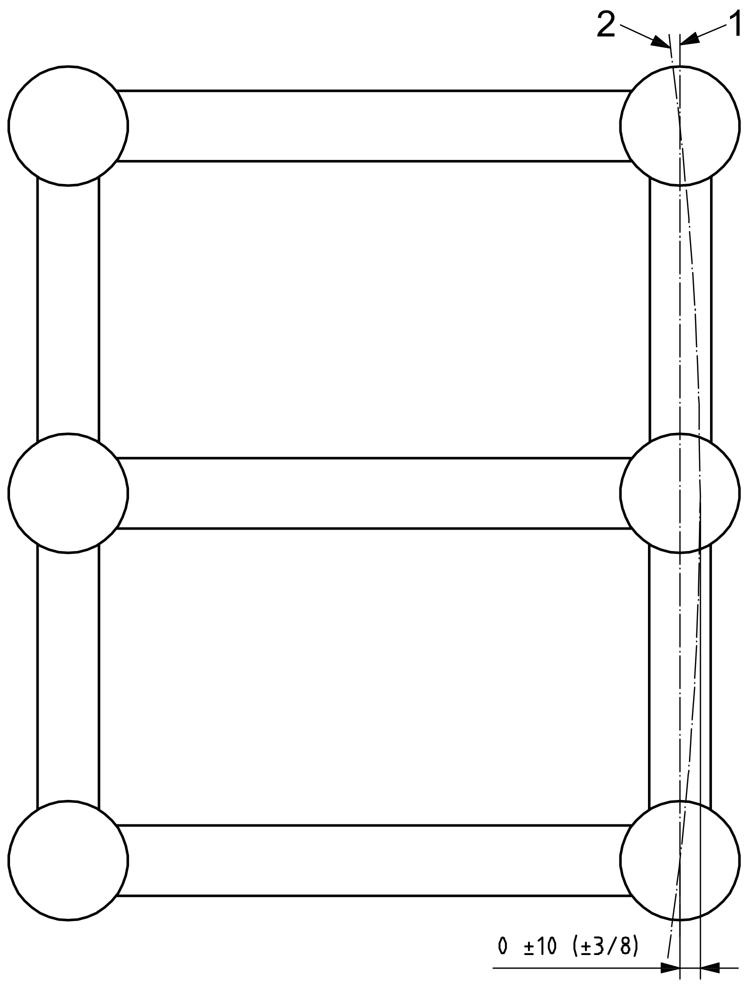
G.9 Leg alignment and straightness tolerances

In addition to the tolerances for individual members (see G.7), the tolerances for the alignment and straightness of legs shall be as shown in Figure G.7 and as follows:

— at each plan bracing level the legs shall be aligned horizontally within 10 mm (3/8 in) of the design location;

— each leg shall be straight to within 10 mm (3/8 in) over the entire length (see Figure G.2).

Dimensions in millimetres (inches)



Key

1 theoretical centreline

2 actual centreline

Figure G.7 — Leg alignment tolerances at plan bracing level (structures with more than 4 legs)

G.10 Tubular joint tolerances

The alignment and tolerances for tubular joints (braces, brace stubs and chords) are shown in Figures G.8 and G.9 and detailed below.

The as-built work point of a brace stub on a chord centreline shall be within 13 mm (1/2 in) of the design work point location.

The as-built work point of a brace on a chord centreline built with point-to-point construction shall be within 20 mm (3/4 in) of the design work point location.

For braces built on the basis of a design minimum gap, g, between brace toes on the chord diameter, the as-built distance between brace toes shall not be less than $g - 3$ mm(1/2 in) or greater than $g + 25$ mm(1 in).

If a joint can or a brace stub contains intermediate circumferential welds the individual lengths within the joint can or brace stub shall be within the range of design length + 10 mm (3/8 in) to design length − 5 mm (1/4 in) and the location of the circumferential welds shall be within 5 mm (1/4 in) of the design location.

The overall lengths of the brace stub of a tubular joint shall be within the range of design length + 50 mm (2 in) to design length − 5 mm (1/4 in).

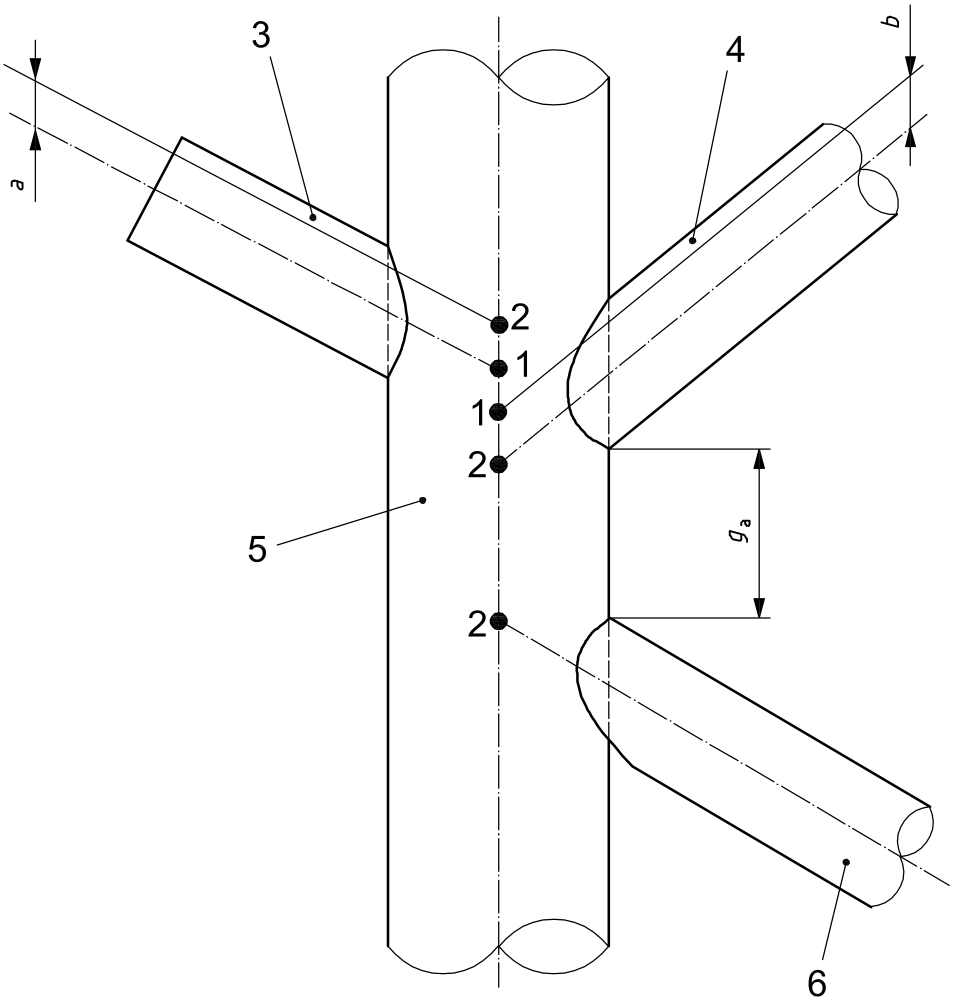
The tolerances on the positioning and alignment of stubs on tubular joints are as follows:

a) the centreline of the brace at the intersection with the chord wall shall be within 5 mm (1/4 in) of the design position where the chord diameter is less than 3,5 m (11,5 ft.);

b) the centreline of the brace at the intersection with the chord wall shall be within 10 mm of the design position where the chord diameter is greater than or equal to 3,5 m (11,5 ft.);

c) the angular orientation of the brace stub shall be within 10 minutes (0,17°) of the design orientation;

d) the position of the centreline of the brace end of the stub shall be within 5 mm (1/4 in), of the design position.



Key

1 design work point

2 as-built work point

3 brace stub

4 brace built point-to-point

5 chord

6 brace built to minimum gap clearance

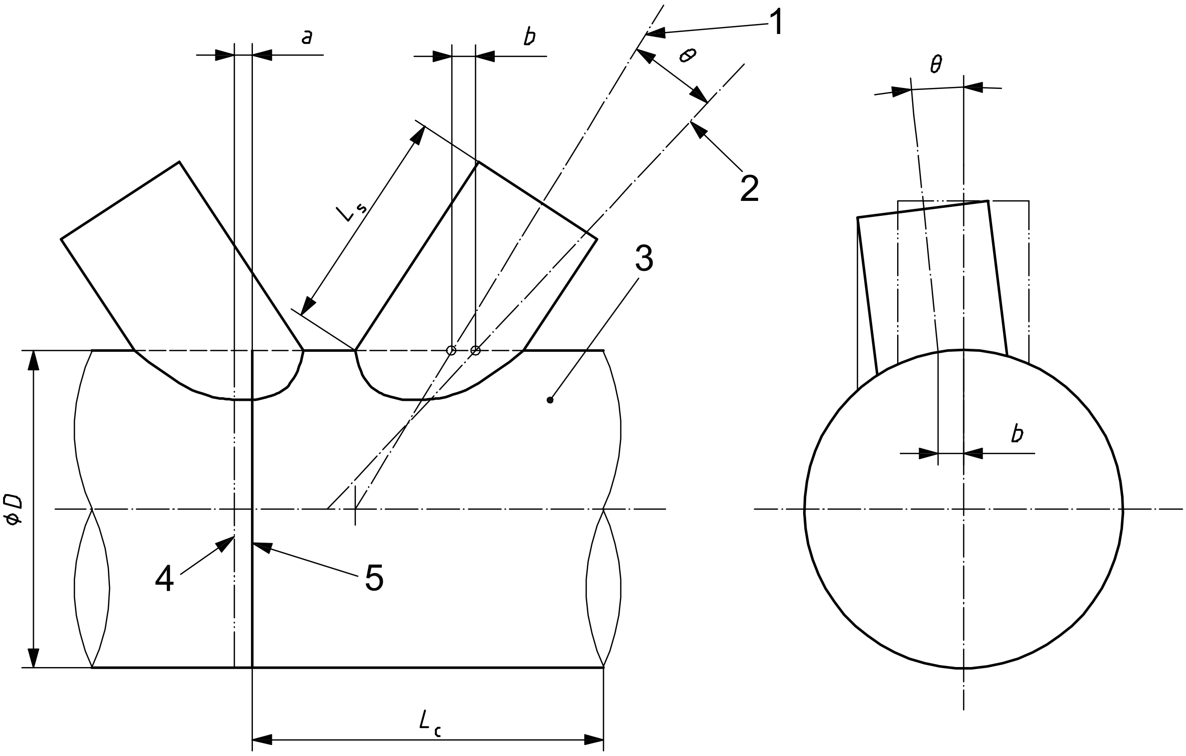
a distance between design and as-built location of brace stub work points on chord centreline $a \leqslant$ 13 mm (0,5 in)

b distance between design and as-built location of brace work points on chord centreline $b \leqslant$ 20 mm (0,75 in)

g_a as-built gap between brace toes

a $g -$ 12 mm (0,5 in) $\leqslant g_a \leqslant g +$ 25 mm (1,0 in), where g is the design gap between brace toes.

Figure G.8 — Tolerances on alignment of brace stubs at tubular joints



for $D < 3{,}5$ m (11,5 ft), $b < 5$ mm (1/4 in)
for $D \geqslant 3{,}5$ m (11,5 ft), $b < 10$ mm (3/8 in)

Key

1 theoretical brace stub centreline
2 actual brace stub centreline
3 node can
4 theoretical circumferential weld
5 actual circumferential weld
6 chord
a offset between actual and theoretical node can position, $a < 5$ mm (1/4 in)
D outside diameter of chord
b offset between actual and theoretical brace centreline intersection at chord outer diameter
L_s theoretical brace stub length
L_c theoretical node can length
θ angle between actual and theoretical centreline of brace, $\theta < 0{,}17°$

[a] $L_s - 5$ mm (0,25 in) $\leqslant L_{s,actual} \leqslant L_s + 50$ mm (2,0 in).

[b] $L_c - 5$ mm (0,25 in) $\leqslant L_{c,actual} \leqslant L_c + 10$ mm (3/8 in).

Figure G.9 — Tolerances for tubular joints

G.11 Cruciform joints

Where plates carrying in-plane forces are arranged to form a cruciform joint, the misalignment shall not exceed the lesser of 50 % of the thickness of the thinnest non-continuous member or 10 mm (3/8 in), whichever is smaller.

G.12 Stiffener tolerances

G.12.1 Stiffener location

Stiffeners shall be positioned to the following tolerances:

a) when within 150 mm (6 in) of a conical transition, accurate to within 3 mm (1/8 in) of the design position;

b) in launch legs, to within 3 mm (1/8 in) of the design position;

c) in tubular joints other than at conical transitions of launch leg nodes, to within 5 mm (1/4 in) of the design position;

d) at all other locations, to within 10 mm (3/8 in) of the design position.

G.12.2 Stiffener cross-section

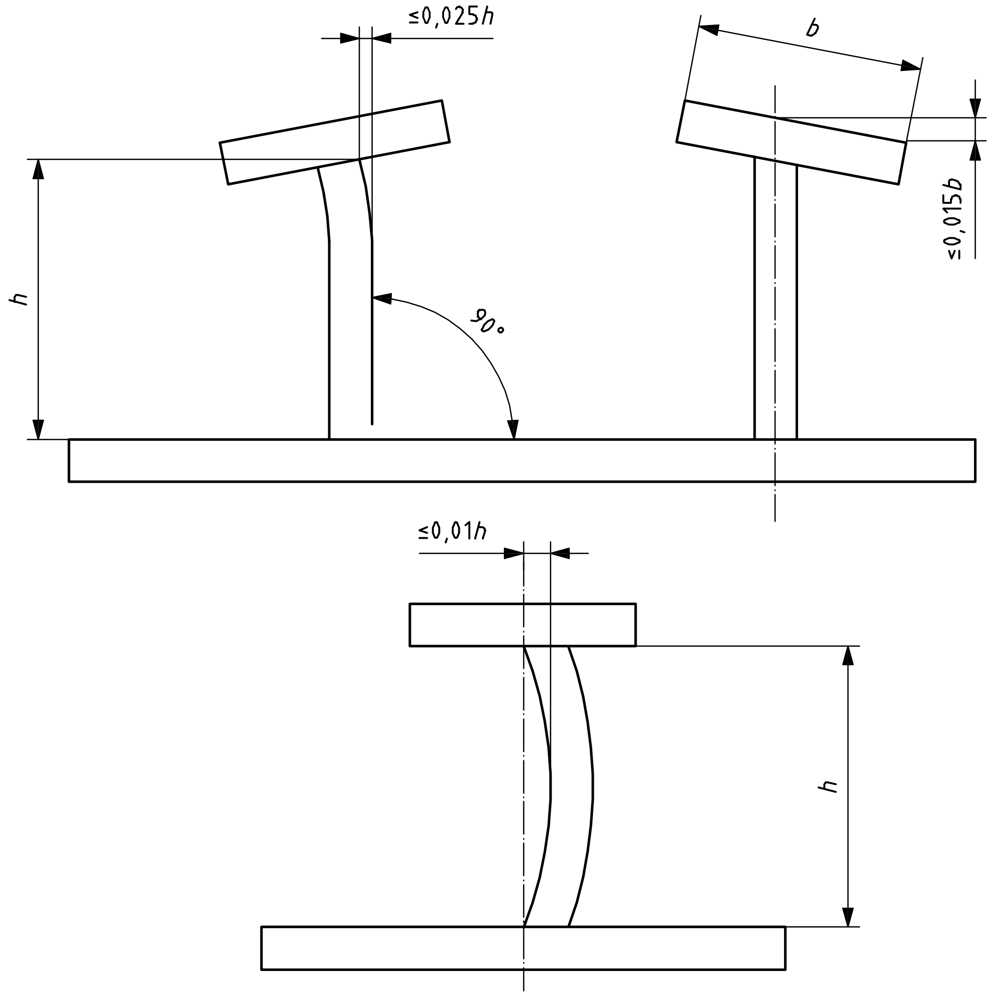
The tolerances on ring stiffener cross-sections are as shown in Figure G.10 and as follows:

a) the web of the stiffener shall be perpendicular to the centreline of the tubular to within 2,5 % of the web height;

b) the flange of the stiffener shall be parallel to the centreline of the tubular to within 1,5 % of the flange width;

c) the web of the stiffener shall be flat over its height to within 1,0 % of the web height.

Out-of-straightness for longitudinal or diaphragm stiffeners in tubular members shall be limited to 0,15 % L or 3 mm, whichever is larger, see Figure G.11.

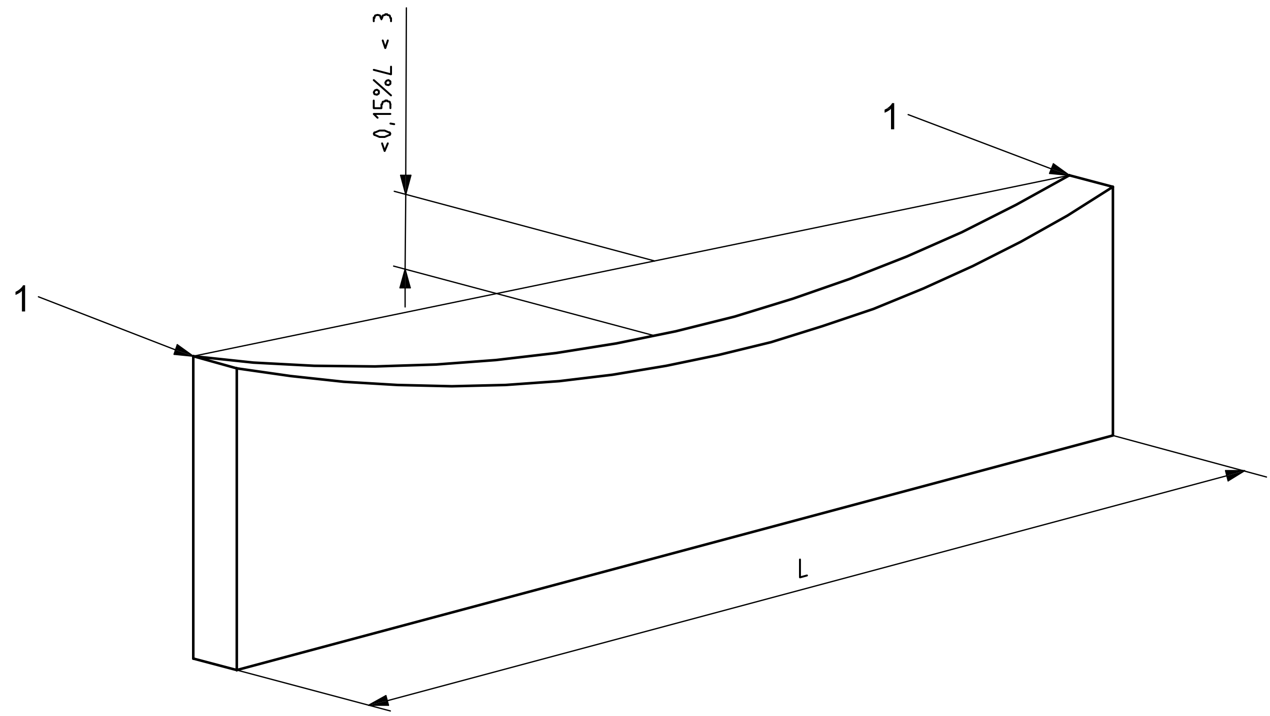


Key

h dent depth

Figure G.10 — Ring stiffener cross-section tolerances

Dimensions in millimetres



Key

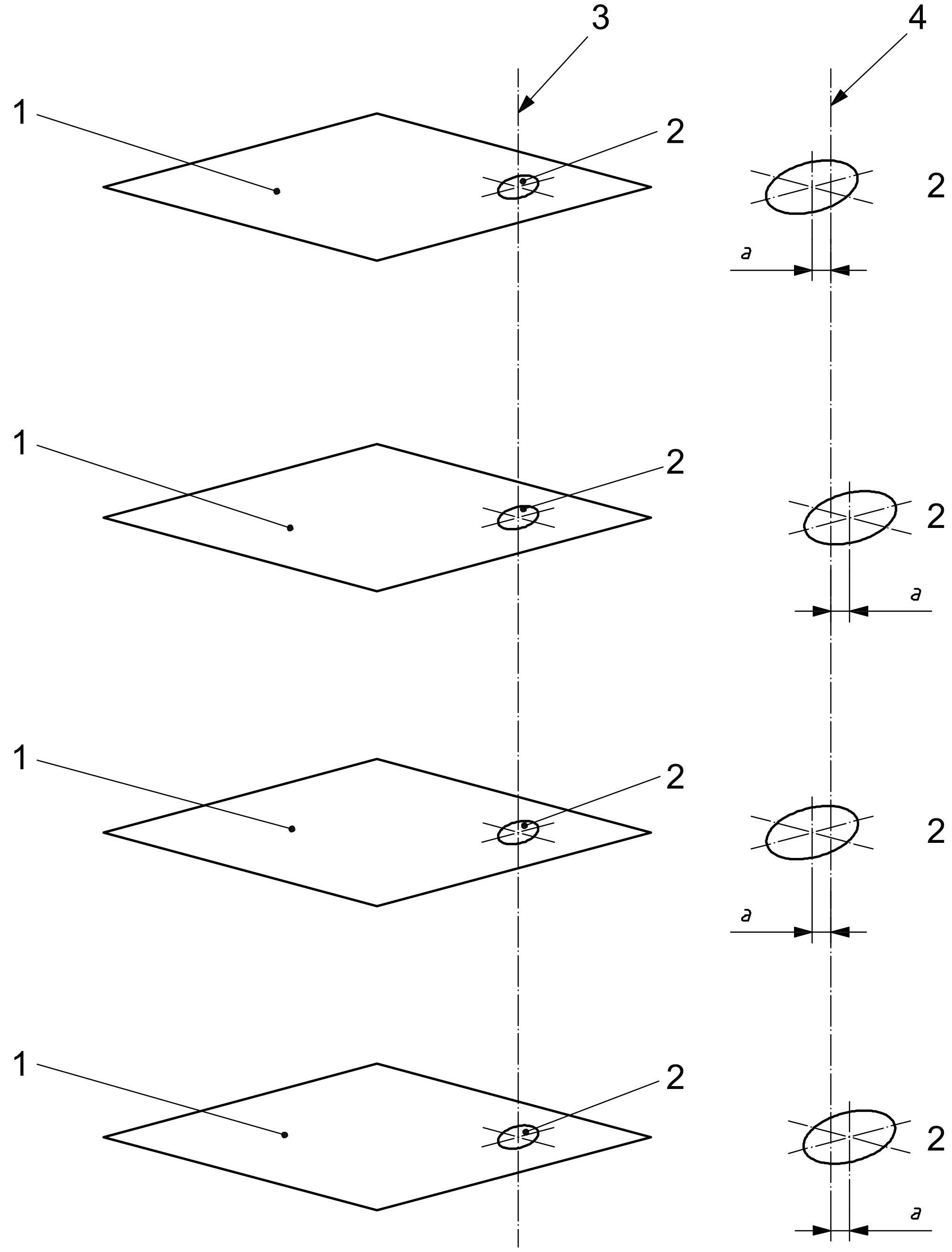
1 point of lateral restraint

Figure G.11 — Ring stiffener straightness tolerance

G.13 Conductor, pile guide, pile sleeve and appurtenance support tolerances

The misalignment on conductor and pile guides, and sleeves and appurtenance supports shall be related to a best-fit line through the centres of the guides, sleeves and supports, see Figure G.12. The tolerance of the centre of each guide/sleeve and the best-fit line shall be less than 10 mm (3/8 in) at the top guide and where the vertical spacing between guides is less than 12 m (40 ft). The tolerance of the centre of each guide/sleeve and the best-fit line shall be less than 13 mm (1/2 in) in other cases. For pile sleeves, the misalignment shall be checked at mid-height of each set of centralizers.

The tolerance of the centre of each appurtenance support and the best-fit line shall be less than 10 mm (3/8 in) at the top and less than 25 mm (1 in) elsewhere.



10 mm (± 3/8 in) $< a <$ +10 mm (± 3/8 in) at top level

10 mm (± 3/8 in) $< a <$ +10 mm (± 3/8 in) for vertical spacing $\leqslant$ 12 m

13 mm (± 1/2 in) $< a <$ +13 mm (± 1/2 in) for vertical spacing $>$ 12 m

Key

1 plan bracing levels

2 centres of conductor guides at each level

3 theoretical centre line

4 best fit centreline

a tolerance of alignment of guides

Figure G.12 — Conductor guide alignment tolerances (also applies to pile guides/sleeves where provided)

Annex H
(informative)

Regional information

H.1 General

This annex contains additional requirements for fixed steel offshore structures applicable to particular geographical regions (which can form all or part of the offshore waters of one or more countries) because of their environments or local custom and practice. The information in this annex is normative for structures located in the defined areas and informative for other regions. The contents have been developed by experts from the region concerned to supplement the provisions of this International Standard. This annex contains some regional and national data, including regional environmental conditions and local design, construction and operating practices. The regulatory framework is explained, but neither regulatory requirements nor reference to specific legislation is included.

H.2 North West Europe

H.2.1 Description of region

The North West Europe waters are diverse and stretch from the sub-Arctic waters off Norway and Iceland to the Atlantic seaboard of France and Eire in the south. Clause H.2 specifically includes the waters of the following countries:

— United Kingdom;

— Eire;

— Norway;

— Germany (North Sea and Baltic Sea);

— Denmark (North Sea and Baltic Sea);

— The Netherlands;

— Belgium;

— France (Atlantic seaboard only);

— Spain (Atlantic seaboard only).

H.2.2 Regulatory framework in NW Europe

H.2.2.1 General

The European Community (EC) countries and European Free Trade Association (EFTA) countries which border the offshore area are linked in the European Economic Area (EEA). Regional requirements are made by the EC. EC requirements have to be implemented by each of the EC members through their own law making process. Under the EEA agreement, EFTA countries have agreed to meet the requirements of directives agreed upon by the EC, which will include directives relating to safety and lifting of trade barriers.

H.2.2.2 UK regulatory framework

The UK Regulatory Framework for oil and gas development is based on a goal setting regime with no detailed technical provisions. It is the responsibility of the duty holder, who is normally the owner, to determine the technical requirements by reference to International Standards, codes of practice or other means.

H.2.2.3 Norwegian regulatory framework

The Norwegian Pollution Control Authority, the Norwegian Social and Health Directorate and the Petroleum Safety Authority (former NPD) have made joint regulations related to health, environment and safety on the Norwegian continental shelf. The regulations are the framework regulations (Royal Decree), the management regulations, the information duty regulations, the facilities regulations and the activities regulations. Guidelines to the regulations have been prepared. The guidelines give references to acceptable standards, codes of practice and other documents.

H.2.2.4 Danish regulatory framework

The Danish Energy Authority is the supervising authority of health and safety. Issues concerning emissions to the environment are handled by the Danish Environmental Protection Agency, while other environmental issues lie with the Danish Energy Authority.

It is the responsibility of the duty holder to ensure that the health and safety level is satisfactory and documented in accordance with the “Act on Safety etc. on Offshore Installations for Exploration, Production and Transportation of Hydrocarbons (The Offshore Safety Act)”.

H.2.2.5 Dutch regulatory framework

The Dutch Regulatory Framework for oil and gas development is based on a goal setting regime with no detailed technical provisions. It is the responsibility of the duty holder to determine the technical requirements by reference to International Standards, codes of practice or other means.

H.2.3 Technical information for NW Europe

H.2.3.1 Partial action factors

The partial action factor for environmental actions shall be 1,35 (see 9.10), unless rational analysis demonstrates an alternative value is appropriate.

H.3 Canada

H.3.1 Description of region

The geographical basis for this annex is the region bounded by the continental shelf margins of Canada. The region encompasses both shallow water and deepwater areas of offshore Canada that are either ice-free regions (Pacific Ocean off the west coast of British Columbia) or regions that may be subjected seasonally to the presence of sea ice and icebergs. Sea ice can be present in the Beaufort Sea, offshore Newfoundland and Labrador, in the Gulf of St. Lawrence, as well as offshore Nova Scotia, although the occurrence of sea ice in the offshore Nova Scotia area is rare. Icebergs are typically encountered in the waters on the north and east coasts of offshore Newfoundland and Labrador.

H.3.2 Regulatory framework in Canada

Oil and gas exploration and production activities in Canada’s non-Accord Frontier Lands (defined as the Northwest Territories, Nunavut Territories, Sable Island and its submarine areas, and areas not within a province adjacent to the coast of Canada, to the outer edge of the continental margin or to a distance of two hundred nautical miles, whichever is greater, but excluding the offshore areas of Nova Scotia and Newfoundland and Labrador) are governed by the Canada Oil and Gas Operations Act[H.1] and the Canada Petroleum Resources Act[H.2]. The Canada Oil and Gas Operations Act and certain elements of the Canada Petroleum Resources Act are administered by the National Energy Board (NEB) in all of the non-Accord Frontier Lands.

Oil and gas exploration and production activities in Canada’s Accord Frontier Lands (defined as offshore areas in the Canada-Nova Scotia Offshore Petroleum Resources Accord Implementation Act (CNSOPRAIA[H.3]) and the Canada-Newfoundland Atlantic Accord Implementation Act (C-NAAIA[H.4]) are governed by the CNSOPRAIA and C-NAAIA and mirror the provincial Accord Implementation Acts, respectively. The provincial acts are the Canada-Nova Scotia Offshore Petroleum Resources Accord Implementation (Nova Scotia) Act [CNSOPRAI(NS)A][H.5] and the Canada-Newfoundland and Labrador Atlantic Accord Implementation Newfoundland and Labrador Act (C-NLAAINLA)[H.6], respectively.These acts are administered by joint federal-provincial offshore petroleum boards. In the Nova Scotia Offshore Accord area the regulator is the Canada-Nova Scotia Offshore Petroleum Board (CNSOPB), and in the Newfoundland and Labrador Offshore Accord area the regulator is the Canada-Newfoundland and Labrador Offshore Petroleum Board (C-NLOPB).

For the offshore areas, the three boards (NEB, CNSOPB, and C-NLOPB) are responsible for the regulation of petroleum activities including

- issuance of licences for offshore exploration and development,
- health and safety of workers,
- protection of the environment during petroleum activities,
- management and conservation of petroleum resources,
- compliance with the provisions of the laws dealing with employment and industrial benefits by the offshore petroleum board in the Accord area, by the Department of Indian Affairs and Northern Development for non-Accord Frontier Lands north of 60° north, and by the Department of Natural Resources for non-Accord Frontier lands south of 60° north, and
- resource evaluation and data collection, curation and distribution.

H.3.3 Technical information for Canada

H.3.3.1 General

Until the publication of ISO 19906[1] on Arctic structures, all requirements for the design of structures for ice and iceberg loads shall be in accordance with CAN/CSA-S471-04[H.7].

H.3.3.2 Partial action factors

The partial action factor for environmental actions shall be 1,35 (see 9.10), unless rational analysis demonstrates that an alternative value is appropriate.

H.3.3.3 Damage due to freezing water

Structural and mechanical systems shall be protected from damage due to freezing water by insulation, heating, drainage, provision of crushable materials, or other suitable means.

H.3.3.4 Dynamic analysis

Dynamic analysis shall properly account for the mechanical characteristics of the total dynamic system. When required, ice-fluid-soil-structure interaction, as well as the dynamic properties of the foundation soil, shall be modelled.

H.3.3.5 Strength of tubular members and joints subjected to ice actions

Structural members and joints that are located in an area where they can be subjected to actions from icebergs or sea ice shall be designed to resist local ice loads combined with relevant global actions.

H.3.3.6 Strength of stiffened plate panels

Uniaxially stiffened plate panels composed of a skin plate, longitudinal stiffeners, and transverse girders shall be designed in accordance with H.3.3.1. The design of stiffened plate panel configurations other than uniaxially stiffened plate panels shall be in accordance with another recognized design standard such as DNV-RP-C201[H.8] or API Bulletin 2V[H.9].

H.3.3.7 Strength of steel-concrete composite walls

Composite ice-resisting walls consisting of external steel plates and concrete infill shall be designed in accordance with H.3.3.1. The requirements of ISO 19903[6] shall govern the placement of concrete and the determination of concrete material properties.

H.3.3.8 Fatigue due to ice-induced cyclic stresses

Cyclic stresses caused by ice loading events shall be included in the fatigue assessment. This includes consideration of high cycle, low amplitude cyclic stresses caused by frequent, low intensity ice loading events, as well as low cycle, high amplitude cyclic stresses caused by rare, high intensity ice loading events.

H.3.3.9 Material toughness requirements

The design class (DC) approach, modified in accordance with this annex, shall be used to determine material toughness requirements.

H.3.3.10 Cathodic protection

For a structure in an area where ice-covered water is common, sacrificial anode systems shall not be used, except on areas of the structure where the anodes are well protected from ice damage. The design of cathodic protection systems shall be based on established methods.

H.3.3.11 Ice abrasion

If the structure can be severely abraded by ice in the splash zone, corrosion protection design shall assume uncontrolled corrosion losses. For structures where ice abrasion will occur, additional wall thickness shall be provided, or another suitable system or combination of systems shall be used to protect the structural members.

H.3.3.12 Knife-line corrosion

The weld metal, HAZ, and parent plate chemistry shall be selected so that metallurgical and electrical potential differences between adjacent metals are minimized in order to avoid selective weld region attack.

H.3.3.13 Welding

If the welding standard selected for use is CSA W59-03[H.10], the additional requirements given in Annexes A, B, and C of CSA-S473-04:2004[H.11], CSA W47.1-03[H.12], CSA W178.1[H.13] and CSAW178.2-01[H.14], shall be used.

H.3.4 Additional information and guidance for Canada

H.3.4.1 Actions and combinations of actions

The Arctic environment requires design considerations besides those associated with similar installations in more temperate areas. These considerations relate to extremely low temperatures and significant actions resulting from iceberg and sea ice interaction with the structure. They include, but are not limited to, high local ice loads, high amplitude dynamic ice loads, and the effect of ice abrasion on corrosion protection.

H.3.4.2 Dynamic analysis

The dynamic effects of actions due to ice, waves, wind, currents, earthquakes, impacts, explosions and functional actions shall be taken into consideration. Dynamic effects are important when the resonant frequencies of a structure or one or more of its components lie in the range where the energy content of the actions can cause a significant increase in load due to dynamic amplification.

Dynamic analysis includes the effects of damping, including internal structural damping of the materials and energy dissipation at the joints.

The water-ice-structure interaction is taken into account by considering the added mass of water and ice, the effects of waves radiating energy to infinity (hydrodynamic damping), and the drag effect and energy dissipation of the ice. If necessary, frequency-dependent impedance functions could be used to represent boundary conditions for complex fluid-solid-structure interactions, including interactions involving ice.

H.3.4.3 Foundation loads on flat-bottom structures

Foundations overlying ice-gouged areas can be subject to non-uniform pressures due to differences in shear strength (short term) and compressibility (long term) of ice-gouge infill and surrounding sea floor sediments.

H.3.4.4 Knife-line corrosion

Experience to date has shown that it is possible to minimize the metallurgical and electrical differences between the weld areas and the parent plates to such a degree that knife-line corrosion is avoided. Various electrical methods have been used to demonstrate that certain levels of potential voltage differences between the different metals correlate well with deterioration rates. Additionally, accelerated corrosion tests can be carried out on welded test specimens to demonstrate that the degree of metallurgical difference in the weld zone (resulting from the welding process) will not result in subsequent high rates of deterioration.

H.3.4.5 Toughness class

Example component classifications are presented in Table H.1 (see also Table D.9).

Table H.1 — Component toughness classification examples

Component	Consequences	Stress state	Design class
Monocone ice structure neck section	Failure could cause loss of entire manned topsides structure	High — thick sections, high restraint	DC1
Skin plating and stiffeners of caisson-type structure with storage of production inventory against outer skin	Failure will not lead to overall collapse; leakage limited to inventory volume	High — plastic design	DC3
Skin plating and stiffeners of caisson-type structure with no oil storage or double hull design with oil storage	Local failure in highly redundant structure; small risk to life and low potential for pollution	High — plastic design	DC5

Bibliography

[1] ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*[5)]

[2] ISO 19901-3, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 3: Topsides structure* [5)]

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

[4] ISO 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups* [5)]

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

[6] ISO 19903, *Petroleum and natural gas industries — Fixed concrete offshore structures*

[7] ISO 19901-7, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units*

[8] ISO 9000, *Quality management systems — Fundamentals and vocabulary*

[9] ISO 9001, *Quality management systems — Requirements*

[10] ISO 9004, *Quality management systems — Guidelines for performance improvements*

[A.7.7-1] European Committee for Standardization (CEN), *Eurocode — Basis of Structural Design*, EN 1990:2002

[A.7.10-1] VAN DE GRAAF, J.W., TROMANS, P.S. and EFTHYMIOU, M. The Reliability of Offshore Structures and its Dependence on Design Code and Environment, *Proc. 27th Offshore Technology Conf.,* OTC Paper 7382, Houston, May 1994

[A.7.10-2] EFTHYMIOU, M., VAN DE GRAAF, J.W., TROMANS, P.S. and HINES, I.M. Reliability Based Criteria for Fixed Steel Offshore Platforms, *Proc. Conf. OMAE-96*, Florence, Italy, June 1996

[A.8.3-1] Lloyds register, *Code for Lifting Appliances in a Marine Environment*, Lloyds, London 2003

[A.9.1-1] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design*, API RP 2A-LRFD, First Edition, 1993

[A.9.5-1] HEIDEMAN, J.C. and WEAVER, T.O. Static wave force procedure for platform design, *Proc. Int. Conf. Civil Eng. In the Oceans V*, ASCE College Station, Texas, 1992, pp. 496-517

[A.9.5-2] MADSEN, O.S., Hydrodynamic Force on Circular Cylinders, *Applied Ocean Research 8*, **No. 3**, 1986

[A.9.5-3] MANNERS, W. and RAINEY, R.C.T. Hydrodynamic Forces on Fixed Submerged Cylinders, *Proc. Royal Society of London*, **436**, 1992

5) Under preparation.

6) To be published.

[A.9.5-4] NEWMAN, J.N., Marine Hydrodynamics, *MIT Press*, 1977

[A.9.5-5] SARPKAYA, T. and ISAACSON, M. *Mechanics of Wave Forces on Offshore Structures*, Van Nostrand Reinhold Co., 1981

[A.9.5-6] KEULEGAN, G.H. and CARPENTER, L.H. Forces on Cylinders and Plates in an Oscillating Fluid, *J. Research of the National Bureau of Standards*, **60**, **No. 5**, May 1958

[A.9.5-7] LAMBRAKOS, K.F., CHAO, J.C., BECKMANN, H. and BRANNON, H.R. Wake Model of Hydrodynamic Forces on Pipelines, *Ocean Engineering*, **14**, **No. 2**, 1987, pp. 117-136

[A.9.5-8] HOERNER, S.F., Fluid Dynamic Drag, Chapter V.1, *Hoerner Fluid Dynamics*, New Jersey, 1965

[A.9.5-9] NORTON, D.J., HEIDEMAN, J.C. and MALLARD, W. Wind Tunnel Tests of Inclined Circular Cylinders, *J. Society of Petroleum Engineers*, **23**, 1983, pp. 191-196

[A.9.5-10] SARPKAYA, T., RAINES, T.S. and TRYTTEN, D.O. Wave Forces on Inclined Smooth and Rough Circular Cylinders, *Proc. 15th Offshore Technology Conf.*, Paper No. 4227, 1982

[A.9.5-11] GARRISON, C.J. Comments on Cross-Flow Principle and Morison's Equation, *ASCE J. Waterway, Port, Coastal, and Ocean Engineering*, **111**, **No. 6**, November 1985

[A.9.5-12] KASAHARA, Y. and SHIMAZAKi, K. Wave Forces Acting on Rough Circular Cylinders at High Reynolds Numbers, *Proc. 20th Offshore Technology Conf.*, OTC 5372, 1987

[A.9.5-13] SCHLICHTING, H. *Boundary Layer Theory*, Chapter XX.g, McGraw-Hill, InC., 1979

[A.9.5-14] NATH, J.H. Hydrodynamic Coefficients for Marine-Roughened Cylinders, *Final Report to API on Practice 85-31,* Department of Civil Engineering, Oregon State University, March, 1987

[A.9.5-15] Det Norske Veritas, *Rules for Classification of Fixed Offshore Installations*, First Edition, 1990

[A.9.5-16] ACHENBACH, E. Influence of Surface Roughness or the Cross-Flow Around a Circular Cylinder, *J. Fluid Mechanics,* Vol. 46, pp. 321-335, 1971

[A.9.5-17] BECKMANN, H. and MCBRIDE, C.M. Inherent Scatter of Wave Forces on Submerged Structures, *Proc. ASME Petroleum Division Joint Conf. with Pressure Vessels and Piping Division,* Dallas, September 22-25, 1968

[A.9.5-18] SARPKAYA, T. *In-Line and Transverse Forces on Smooth and Rough Cylinders in Oscillatory Flow at High Reynolds Numbers*, Naval Postgraduate School, Report NPS69-86-003, July 4, 1986

[A.9.5-19] HEIDEMAN, J.C., OLSEN, O. and JOHANSSON, P. Local Wave Force Coefficients, *Proc. Civil Engineering in the Oceans IV Conf.*, ASCE, September 1979

[A.9.5-20] STEELE, K.M. Performance of the Lena Guyed Tower, *Proc. 19th Offshore Technology Conf.,* OTC 5255, 1986

[A.9.5-21] MONOPOLIS, G.M. and DANACZKO, M.A. Installation Model Tests of a Gulf of Mexico Compliant Tower, *Proc. 22nd Offshore Technology Conf.,* OTC 5911, 1989

[A.9.5-22] TAYLOR, P.H. Current Blockage: Reduced Forces on Offshore Space-Frame Structures, *Proc. 24th Offshore Technology Conf.*, OTC 6519, 1991

[A.9.5-23] FORRISTALL, G.Z. Measurements of Current Blockage by the Bullwinkle Platform, *J. Atmospheric and Oceanic Technology*, pp. 247-1266, Vol 13, No. 6, December 1996

[A.9.5-24] LAMBRAKOS, K. F. and BECKMANN, H. Shielding and Interference Model for Offshore Platforms. *Procs. 5th Intl. Conf. on the Behaviour of Offshore Structures BOSS 1992*, London, July 1992

[A.9.5-25] HEIDEMAN, J.C. and SARPKAYA, T. Hydrodynamic Forces on Dense Arrays of Cylinders, *Proc. 18th Offshore Technology Conf.*, OTC Paper 5008, 1985

[A.9.5-26] REED, K., AARSNES, J.V., BELTRAND, O. and ANDERSEN, E. Wave and Current Forces on Conductor Pipe Groups, *Society for Underwater Technology, Proc. Environmental Forces on Offshore Structures and Their Prediction Conf.*, 1990

[A.9.7-1] ENV 1991-2-4, *Basis of design and actions on structures —Part 2-4: Actions on structures — Wind Actions,* April, 2000

[A.9.8-1] WHEELER, J.D. Method for prediction of forces produced by irregular waves, *J. Petroleum Tech.*, March 1970, pp. 359-367

[A.9.8-2] RODENBUSCH, G. and FORRISTALL, G.Z. An empirical model for random directional wave kinematics near the free surface, *Proc. 18th Offshore Technology Conf.*, OTC 5097, Houston, Texas, 1986

[A.9.8-3] SHARMA, J.N. *Development and evaluation of a procedure for simulating a random, directional sea surface and associated wave forces*, (PhD Thesis), University of Dalaware, 1979

[A.9.8-4] RODENBUSCH, G. and KALLSTROM, C. Forces on a large cylinder in random two-dimensional flows, *Proc. 18th Offshore Technology Conf.,* OTC 5096, Houston, May, 1986

[A.9.9-1] ELLINGWOOD, B.B., GALAMBOS, T.V., MACGREGOR, J.G. and CORNELL, C.A. *Development of a Probability Based Load Criterion for American National Standard A58 Building Code Requirements for Minimum Design Loads in Buildings and Other Structures*, National Bureau of Standards Special Publication 577, June 1980

[A.9.9-2] American National Standards Institute, *Building Code Requirements for Minimum Design Loads for Buildings and Other Structures*, ANSI A58.1-1982

[A.9.9-3] MOSES, F. *Guidelines for Calibrating API RP 2A for Reliability-Based Design*, API-PR.C 80-22 Project Report, October 1981

[A.9.9-4] MOSES, F. *Utilising a Reliability-Based API RP2A Format*, API-PR.C 82-22 Project Report, November 1983

[A.9.9-5] MOSES, F. and RUSSELL, L. *Applicability of Reliability Analysis in Offshore Design Practice*, API-PR.C 79-22 Project Report, June 1980

[A.9.9-6] MOSES, F. *Development of Preliminary Load and Resistance Design Document for Fixed Offshore Platforms*, API-PR.C 85-22 Project Report, January 1986

[A.9.9-7] MOSES, F. *System Reliability Developments in Structural Engineering*, Structural Safety, Vol. 1, 1982, pp. 3-13

[A.9.9-8] VAN DE GRAAF, J.W., TROMANS, P.S. and EFTHYMIOU, M. The Reliability of Offshore Structures and its Dependence on Design Code and Environment, *Proc. 27th Offshore Technology Conf.*, Paper 7382, Houston, May 1994

[A.9.9-9] EFTHYMIOU, M., VAN DE GRAAF, J.W., TROMANS, P.S. and HINES, I.M. Reliability Based Criteria for Fixed Steel Offshore Platforms, *Proc. Conf. OMAE-96*, Florence, Italy, June 1996

[A.9.9-10] Bomel Ltd., *System-based calibration of north-west European Annex Environmental Load Factors to ISO Fixed Structures Code 19902*. Report No. C925\04\016R, **Rev B**, November 2002

[A.9.9-11] Bomel Ltd., *Component-based calibration of north-west European Annex Environmental Load Factors to ISO Fixed Structures Code 19902*. Report No. C925\04\017R, **Rev B**, November 2002

[A.9.9-12] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Working Stress Design*, API RP 2A-WSD, 18th Edition, 1993

[A.9.9-13] DIGRE, K.A., PUSKAR, F.J., AGGARWAL, R.K., IRICK, J.T., KRIEGER, W. F. and PETROUSKAS, C. Modifications to and Applications of the Guidelines for Assessment of Existing Platforms Contained in Section 17 of API RP 2A*, Proc. 28th Offshore Technology Conf.*, OTC 7779, Houston, 1995

[A.9.9-14] MOSES, F. *Implementation of a reliability based API RP 2A Format*, API-PRAC 83-22 Project Report, January 1985

[A.9.10-1] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Working Stress Design*, API RP 2A-WSD, 18th Edition, 1993

[A.9.10-2] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design*, API RP 2A-LRFD, First Edition, 1993

[A.9.10-3] MOSES, F. *Utilising a Reliability Based API RP 2A Format*, API-PR.C 82-22 Project Report, November 1983

[A.9.11-1] RODENBUSCH, G. and GUTIERREZ, C.A. *Forces on Cylinders in Two-Dimensional Flows*, Shell Development Company, Report No BRC-13-83, **1**, 1983

[A.9.11-2] SARPKAYA, T., *In-Line and Transverse Forces on Smooth and Rough Cylinders in Oscillatory Flow at High Reynolds Numbers,* Naval Postgraduate School, Report NPS69-86-003, July 4, 1986

[A.9.11-3] NORTON, D.J., HEIDEMAN, J.C. and MALLARD, W. Wind Tunnel Tests of Inclined Circular Cylinders, *J. Soc. Petroleum Engineers*, **23**, 1983, pp. 191-196

[A.9.11-4] SARPKAYA, T. and ISAACSON, M. *Mechanics of Wave Forces on Offshore Structures*, Van Nostrand Reinhold Co., 1981

[A.9.11-5] RIDLEY, J.A. *A study of some theoretical aspects of slamming*, NMI R158, OT-R-8482113, 1982

[A.9.11-6] BARLTROP, N.D.P., MITCHELL, G.M. and ATTKINS, J.B. *Fluid Loading on Fixed Offshore Structures*, Atkins Engineering Sciences, HSE, OTH 90 322, London, 1990

[A.10.4-1] ISO 17776, *Petroleum and natural gas industries — Offshore production installations — Guidelines on tools and techniques for hazard identification and risk assessment*

[A.10.4-2] American Petroleum Institute, *Recommended Practice for the Design of Offshore Facilities Against Fire and Blast Loading*, API RP 2FB, First Edition

[A.10.4-3] UK Offshore Operators Association (UKOOA), *Fire and Explosion Guidance, Part 0, Fire and Explosion Hazard Management*, October 2003

[A.10.4-4] UK Offshore Operators Association (UKOOA), *Fire and Explosion Guidance, Part 1, Avoidance and mitigation of explosions*, October 2003

[A.10.4-5] UK Offshore Operators Association (UKOOA), *Fire and Explosion Guidance, Part 2, Avoidance and mitigation of fires*, (in preparation)

[A.10.4-6] UK Offshore Operators Association (UKOOA), *Fire and Explosion Guidance, Part 3, Detailed design and assessment guidance*, (in preparation)

[A.10.4-7] Norsok Standard Z-013, *Risk and emergency preparedness analysis*, Rev. 2, 2001-09-01

[A.11.6-1] MARSHALL, P.W. and CHANG, B. Structural design considerations, *Proc. 29th Offshore Technology Conf.*, Houston, Texas, Paper OTC 8107, May 1996

[A.12.2-1] DNV-SINTEF-BOMEL, *Ultiguide — Best practice guidelines for use of non-linear analysis methods in documentation of ultimate limit states for jacket type offshore structures*, DNV, April 1999

[A.12.3-1] BARLTROP, N.D.P., CULVERHOUSE, A. and MCLOUGHLIN, M.J., *Node flexibility and its effect on jacket structures — A pilot study on 2 dimensional frames*, Report UR22 UEG, CIRIA. ISSN: 0305 4055, ISBN 0 86017 213 9, 1984

[A.12.3-2] BARLTROP, N.D.P. and ADAMS, A.J. *Dynamics of fixed marine structures*, MTD/Butterworths, 1991

[A.12.5-1] TROMANS, P.S. and VAN DE GRAAF, J.W. A substantiated risk assessment of a jacket structure, *Proc. 25th Offshore Technology Conf.*, Houston, Texas, Paper OTC 7075, May 1992

[A.12.5-2] SALMON, C.G. and JOHNSON, J.E. *Steel structures design and behaviour*, Second Edition, Harper & Row Publishers, New York, 1980

[A.12.5-3] TALL, L. and ALPSTEN, G.A. On the scatter in yield strength and residual stresses in steel members, *Symp. on Concepts of Safety of Structures and Methods of Design*, London, 1969

[A.12.5-4] BAKER, M.J. *Variability in strength of structural steels — A study in structural safety — Part 1: Material variability*. Technical Note 44, CIRIA, September 1972

[A.12.5-5] BEEDLE, L.S. and TALL, L. Basic column strength. *J Structural Division*, ASCE, Volume 86, No ST7, Paper 2555, July 1960

[A.12.5-6] Joint Industry Project, *Hurricane Andrew* — Effects on offshore platforms, PMB Engineering Inc. San Francisco CA 94111, October 1993

[A.12.5-7] VAN LANGEN, H. et al. Integrated foundation and structure reliability analysis of a North Sea structure, *Proc. 28th Offshore Technology Conf.*, Houston, Texas, Paper OTC 7784, May 1995

[A.13.1-1] Det Norske Veritas, *Buckling Strength Analysis of Shells,* Recommended Practice DNV-RP-C202*,* October 2002

[A.13.1-2] American Petroleum Institute, *Stability Design of Cylindrical Shells*, Bulletin 2U*,* Second Edition, 2000

[A.13.2-1] MILLER, C.D. and SALIKLIS, E.P. *Analysis of Cylindrical Shell Database and Validation of Design Formulations*, API PRAC Project 90-56, October 1993

[A.13.2-2] AME, *Derivation of Buckling Safety Factors for Offshore Structural Components*, Offshore Technology Report OTN 94 209, January 1995

[A.13.2-3] American Petroleum Institute, *Specification for the Fabrication of Structural Steel Pipe*, API Spec 2B, Sixth Edition

[A.13.2-4] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design*, API RP 2A-LRFD, First Edition, 1993

[A.13.2-5] CHEN, W.F. and ROSS, D.A. Tests of Fabricated Tubular Columns, *J. Structural Division*, ASCE, Vol. 103, No. ST3, March 1977

[A.13.2-6] SMITH, C.S., SOMERVILLE, W.C. and SWAN, J.W. Residual Strength and Stiffness of Damaged Steel Bracing Members, *Proc. 14th Offshore Technology Conf.,* OTC Paper No. 3981, Houston, Texas, 4-7 May 1981

[A.13.2-7] STEINMANN & VOJTA, J.F. *Hydrostatic Beam Column Tests, (Phase II)*, Final Report, CBI Technical Service Co., API Project No. 87-56, February 1989

[A.13.2-8] YEOMANS, N.F. *Buckling Strength of Hot-Finished CHS,* CIDECT program 2D, CIDECT Report 76/40, 1976

[A.13.2-9] SMITH, C.S., KIRKWOOD, W. and SWAN, J.W. Buckling Strength and Post-Collapse Behaviour of Tubular Bracing Members Including Damage Effects, *Procs. 2nd Intl. Conf. on the Behaviour of Offshore Structures*, BOSS 1979, London, August 1979

[A.13.2-10] MARZULLO, M.A. and OSTAPENKO, A. Tests on Two High-Strength Short Tubular Columns, *Proc. Offshore Technology Conf.,* OTC Paper No. 3086, Houston, Texas, May 1978

[A.13.2-11] OSTAPENKO, A. and GRIMM, D.F*., Local Buckling of Cylindrical Tubular Columns Made of A36 Steel*, Report No. 450-7, Fritz Engineering Laboratory, Lehigh University, February 1980

[A.13.2-12] PRION, H.G.L. and BIRKEMOE, P.C. *Experimental Behaviour of Unstiffened Fabricated Tubular Steel Beam-Column*, Report No. 88-03, Dept. Of Civil Engineering, University of Toronto, Ontario, Canada1988

[A.13.2-13] EDER, M.F., GROVE, R.B., PETERS, S.W. and MILLER, C.D. *Collapse Tests of Fabricated Cylinders Under Combined Axial Compression and External Pressure*, Final Report, American Petroleum Institute PRAC Project 82/83-46, February 1984

[A.13.2-14] KIZILTUG, A.Y., GROVE, R.B., PETERS, S.W. and MILLER, C.D. *Collapse Tests of Short Tubular Columns Subjected to Combined Loads*, Final Report to Joint Industry Group, CBI Industries, Inc., December 1985

[A.13.2-15] Norwegian Petroleum Directorate, *Guidelines on Design and Analysis of Steel Structures in the Petroleum Activity*, 1994

[A.13.2-16] SHERMAN, D.R. *Bending Capacity of Fabricated Pipe at End Connections*, API Project No. 5428.09, Civil Engineering Department, University of Wisconsin-Milwaukee, April 1987

[A.13.2-17] KOROL, R.M. and HUDOBA, J. Plastic Behaviour of Hollow Structural Sections, *Proc. Structural Division,* ASCE, 98, ST5, 1007-1023, 1972

[A.13.2-18] KOROL, R.M. Critical Buckling Strains of Round Tubes in Flexure, *Int. J. Mech. Sci.* **21**, 1979, pp. 719-730

[A.13.2-19] SHERMAN, D.R. *Comparative Study of Flexural Capacity of Pipes,* Final Report API Project No. 54201.11, Civil Engineering Department, University of Wisconsin-Milwaukee, March 1984

[A.13.2-20] MILLER, C.D., KINRA, R.K. and MARLOW, R.S. Tension and Collapse Tests of Fabricated Steel Cylinders, *Proc. Offshore Technology Conf.,* Paper No. OTC 4218, Houston, Texas, May 1982

[A.13.2-21] MILLER, C.D. and KINRA, R.K. External Pressure Tests of Ring Stiffened Fabricated Steel Cylinders, *Proc. Offshore Technology Conf.,* Paper No. OTC 4107, Houston, Texas, May 1981

[A.13.2-22] American Institute of Steel Construction, *Manual of Steel Construction — Allowable Stress Design*, Ninth Edition, 1989

[A.13.2-23] BS EN 1993-1-1, *Eurocode 3: Design of Steel Structures — Part 1.1: General Rules and Rules for Buildings*, BSI 2005 (also available from other CEN members)

[A.13.2-24] Norwegian Standard NS 3472, *Design of Steel Structures*, NBR 1984

[A.13.3-1] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Working Stress Design*, API RP 2A-WSD, 20th Edition, 1993

[A.13.3-2] ELLIS, J.S. Plastic Behaviour of Compression Members, *J. Mechanics and Physics of Solids*, **6**, 1958, pp. 282-300

[A.13.3-3] WAGNER, A.L., MUELLER, W.H. and ERZURUMLU, H. Ultimate Strength of Tubular Beam-Columns, *Proc. ASCE*, Structural Division 103, ST1, 1977, pp. 9-12

[A.13.3-4] KATO, B. and AKIYAMA, H. A Proposal for Design Formulae of Beam-Columns, ECCS, *2nd Int. Colloquium on Stability*, Tokyo, Japan, 9 September 1976, pp. 41-60

[A.13.4-1] LOH, J.T. *A Unified* Design Procedure for Tubular Members, *Proc., Offshore Technology Conf.*, No. 6310, Houston, May 1990

[A.13.4-2] LOH, J.T. *A Unified Design Procedure for Tubular Members*, Exxon Production Research Co., Offshore Div., Aug. 20, 1989

[A.13.4-3] KINRA, R.K. Stability under Hydrostatic Pressure and Axial Tension, *Proc., SSRC Annual Technical Meeting and Meeting*, pp. 132-142, New Orleans, 1982

[A.13.4-4] STAHL, B. and BAUR, M.P. Design Methodology for Offshore Platform Conductors, *Proc., Offshore Technology Conf.*, No. 3902, Houston, May 1980

[A.13.5-1] HELLAN, O., MOAN, T. and DRANGE, S.O. Use of Non-Linear Pushover Analyses in Ultimate Limit State Design and Integrity Assessment of Jacket Structures, *Procs. 7th Intl. Conf. on the Behaviour of Offshore Structures,* BOSS 1994, Cambridge, Massachusetts, 1994

[A.13.5-2] KNAPP, A.E. and DIXON, D.A. The Use of X-Bracing in Fixed Offshore Platforms, *Proc. 5th Offshore Technology Conf.,* OTC Paper No. 1663, Houston, 1972

[A.13.5-3] LIVESLEY, R.K. The Application of an Electronic Digital Computer to Some Problems of Structural Analysis, *The Structural Engineer*, January 1956

[A.13.5-4] BUITRAGO, J., HEALY, B.E. and CHANG, T.Y. Local Joint Flexibility of Tubular Joints, *Offshore Mechanics and Arctic Engineering Conf.*, OMAE, Glasgow, UK, 1993

[A.13.5-5] LOH, J.T*. Interpretive Assessment of X-Brace Frame Test Results*, Exxon Production Co., Houston, Texas, June 21, 1990

[A.13.5-6] GALAMBOS, T.V. (ed.), *Guide to Stability Design Criteria for Metal Structures*, Fourth Edition, Wiley & Sons, 1988

[A.13.5-7] American Institute of Steel Construction (AISC), *Manual of Steel Construction — Load and Resistance Factor Design*, **1**, Second Edition, 1994

[A.13.6-1] BOARDMAN, H.C. Stresses at Junctions of Two Right Cone Frustrums with a Common Axis, *The Water Tower,* Chicago Bridge and Iron Company, March 1948

[A.13.7-1] SMITH, C.S. Assessment of Damage in Offshore Steel Platforms, *Procs. Int. Conf. on Offshore and Marine Safety*, Glasgow, September 1983

[A.13.7-2] TABY, J. and MOAN, T. Collapse and Residual Strength of Damaged Tubular Members, *Proc. Fourth Int. Conf. on Behaviour of Offshore Structures*, July 1985

[A.13.7-3] TABY, J. and MOAN, T. Ultimate Behaviour of Circular Tubular Members with Large Initial Imperfections, *Procs., 1987 Annual Technical Session*, Structural Stability Research Council, March 1987

[A.13.7-4] SMITH, C.S. Residual Strength of Tubulars Containing Combined Bending and Dent Damage, *Procs., Offshore Operations Symp., Ninth Annual Energy Sources Tech., Conf. and Exhb.*, New Orleans, Louisiana, February 1986

[A.13.7-5] LOH, J.T., KAHLICH, J.L. and BROEKERS, D.L., *Dented Tubular Steel Members*, Internal Report, Exxon Production Co., Houston, Texas, March 31, 1992

[A.13.7-6] FRIEZE, P.A. Impact Damage and Assessment of Offshore Tubulars, *Proc. 26th Offshore Technology Conf.,* OTC 7152, Houston, May 1993

[A.13.7-7] OSTAPENKO, A. et. al., *Residual Strength of Damaged and Deteriorated Tubular Members in Offshore Structures*, **1**, ATLSS Report No. 93-03, January 1993

[A.13.7-8] RICLES, J.M. Experimental and Analytical Studies on the Residual Strength and Repair of Dent-Damaged Platform Bracing, *Proc. 27th Offshore Technology Conf.,* OTC 7500, Houston, May 1994

[A.13.7-9] LANDET, E. and LOTSBERG, I. Laboratory Testing of Ultimate Strength of Dented Tubular Members, ASCE, *J. of Structural Engineering*, Vol. 118, No. 4, April 1992

[A.13.7-10] YAO, T., TABY, U. and MOAN, T. Ultimate Strength and Post-Ultimate Strength Behaviour of Damaged Tubular Members in Offshore Structures, *Proc. Intl. Symp. on Offshore Mechanics & Arctic Engineering*, ASME, Tokyo, 1986

[A.13.7-11] ALLEN, J. and MARSHALL, J. *Ship Impact on Steel Tubulars*, University of Strathclyde, Report to Department of Energy, OTI 88 532, 1988

[A.13.7-12] ALLEN, J. and MARSHALL, J. *The Effect of Ship Impact on the Load Carrying Capacity of Steel Tubulars*, University of Strathclyde, Report to Health and Safety Executive, OTH 90 317, 1992

[A.13.7-13] RICLES, J.M., BRUIN, W.M., SOOI, T.K., GEBOR, M.F. and SCHONWETTER, P.C. Residual Strength Assessment and Repair of Damaged Offshore Tubulars, *Proc. 28th Offshore Technology Conf.*, OTC Paper 7807, Houston, 1995

[A.13.7-14] LOH, J.T. Ultimate Strength of Dented Tubular Steel Members, *Proc. Third Intl. Offshore and Polar Engineering Conf.*, Singapore, 1993

[A.13.7-15] UEDA, Y. and RASHED, S.M. Behaviour of Damaged Tubular Members, *Proc., Fourth Int. OMAE Symp.*, Dallas, Texas, 1985

[A.13.8-1] OSTAPENKO, A. et al., *Residual Strength of Damaged and Deteriorated Tubular Members in Offshore Structures*, ATLSS Engineering Research Centre, ATLSS Report No. 93-03, Lehigh University, Bethlehem, Pennsylvania, January 1993

[A.13.8-2] RICLES, J.M. and HEBOR, M.F. Residual Strength and Epoxy-Based Grout Repair of Corroded Offshore Tubulars, *Procs. 7th Intl. Conf. on the Behaviour of Offshore Structures,* BOSS 1994, July 1994

[A.13.8-3] OSTAPENKO, A. et al., *Corrosion Damage — Effect on Strength of Tubular Columns with Patch Corrosion*, ATLSS Engineering Research Centre, ATLSS Report No. 96-01, Lehigh University, Bethlehem, Pennsylvania, June 1996

[A.13.9-1] LOH, J.T. *Grout-Filled Undamaged and Dented Tubular Steel Members*, Exxon Production Research Co., Houston, Texas, May 1992

[A.13.9-2] MSL Engineering, Joint Industry Project: *Strengthening, Modification and Repair of Offshore Structures*, Final Report, MSL, Ascot, Doc Ref. C11100R222 Rev. 1, September 1995

[A.13.9-3] Department of Energy, *Residual and Fatigue Strength of Grout-Filled Damaged Tubular Members*, HMSO, Offshore Technology Report, OTH 89 314, London, 1990

[A.13.9-4] PARSANEJAD, S. Strength of Grout Filled Tubular Members, *J. Structural Division*, ASCE Vol. 113(3), March 1987

[A.13.9-5] Department of Energy, *Grouted and Mechanical Strengthening and Repair of Tubular Steel Offshore Structures*, HMSO, Offshore Technology Report No. OTR 88 283, London, 1988

[A.13.9-6] PARSANEJAD, S., TYTER, S. and CHIN, Y. Experimental Investigation of Grout Filled Damaged Tubular Members, *Steel and Aluminium Structures Conf.*, University College, Cardiff, 1987

[A.13.9-7] TWENTYMAN, N.J. *The Residual Strength of Damaged Tubular Members, Partially Filled with Grout*, Report for Wimpey Group Services, City University, London, July 1989

[A.13.9-8] RICLES, J.M., GILLUM, T.E. and LAMPORT, W.B. Grout Repair of Dent-Damaged Steel Tubular Bracing, *Offshore Technology Conf.*, OTC Paper No. 7151, Houston, Texas, 1993

[A.13.9-9] British Standards Institution (BSI), *Steel, Concrete, and Composite Bridges — Part 5. Code of Practice for Design of Composite Bridges*, BS 5400-5, London, 1979

[A.13.9-10] BASU, A.K. and GHOSH, S.K. *Strength Calculation for Concrete-Filled Tubular Columns of Circular Cross-section*, CIRIA Tech. Note 58*,* U.K. 1975

[A.13.9-11] VIRDI, K.S. and DOWLING, P.J. A Unified Design Method for Composite Columns, *Proc. IABSE*, **36**, Zurich, 1976

[A.14.1-1] BAKER, M.J. *Variability of the Strength of Structural Steel — A Study in Structural Safety*, CIRIA Technical Note No. 44, London, April 1973

[A.14.2-1] MSL Engineering Limited, *Assessment Criteria, Reliability and Reserve Strength of Tubular Joints*, Doc. Ref. C14200R018, Ascot, England, March 1996

[A.14.2-2] DIER, A.F. and LALANI, M. New code formulations for tubular joint static strength, *Proc. 8th Intl, Symp. On Tubular Structures,* Singapore, August 1998

[A.14.2-3] LALANI, M., NICHOLS, N.W. and SHARP, J.V. The Static Strength and Behaviour of Joints in Jack-Up Rigs, *Conf. on Jack-up rigs*, City University, London, August 1993

[A.14.2-4] BOMEL Limited. *Static Strength of High Strength Steel Tubular Joints*. Doc. Ref. C753\01\009R, Maidenhead, England, February 1999

[A.14.2-5] BUITRAGO, J. et al. Local Joint Flexibility of Tubular Joints, *Proc. 12th Intl. Conf. on Offshore Mechanics and Arctic Engineering*, Glasgow, 1993

[A.14.2-6] DIER, A.F. and HELLAN, O. A non-linear tubular response model for push-over analysis. *Proc. 21st OMAE Conf.Intl.*, Oslo, June 2002

[A.14.2-7] CONNELLY, L.M. and ZETTLEMOYER, N. Hydrostatic Pressure Effects on the Capacity of DT Tubular Joints, *Proc. 24th Offshore Technology Conf.,* OTC 6574, Houston, May 1991. OTC 6574, Houston, May 1991

[A.14.2-8] ZETTLEMOYER, N. Developments in Ultimate Strength Technology for Simple Tubular Joints, *Proc. of Conf. on Recent Developments in Tubular Joint Technology*, OTJ 1988, Surrey, U.K., October 1988

[A.14.2-9] American Welding Society, *American Welding Society's D1.1/D1.1M Structural Welding Code — Steel*, 2004

[A.14.2-10] LEE, M.M.K. and DEXTER, E.M. *A New Capacity Equation for Axially Loaded Multi-planar Tubular Joints in Offshore Structures*, HSE, OTO Report 1999095, December 1999

[A.14.2-11] WARDENIER, J., KUROBANE, Y., PACKER, J.A., DUTTA, D. and YEOMANS, N. *Design Guide for Circular Hollow Section (CHS) Joints under Predominantly Static Loading*, CIDECT. Verlag TUV, Rheinland, Germany, 1991

[A.14.2-12] VAN DER VEGTE, G.J. et al. The Static Strength and Stiffness of Multi-planar Tubular Steel X-Joints, *ISOPE J.*, **1, No. 1**, March 1991

[A.14.2.13] PAUL, J.C. et al. Ultimate Resistance of Tubular Double T-Joints under Axial Brace Loading, *J. Construct Steel Research*, **24**, 1993

[A.14.2-14] PAUL, J.C. et al. Ultimate Behaviour of Multi-planar Double K-Joints of Circular Hollow Section Members. *ISOPE J.*, **3**, March 1993

[A.14.3-1] HEALY, B.E. and ZETTLEMOYER, N. In-Plane Bending Strength of Circular Tubular Joints, *Proc. 5th Intl. Symp. on Tubular Structures*, Nottingham University, 1993

[A.14.3-2] PECKNOLD, D.A., MARSHALL, P.W. and BUCKNELL, J.R. New API RP 2A Tubular Joint Strength Design Provisions. *Proc. of 38th Offshore Technology Conf.*, OTC 17295, Houston, May 2005

[A.14.3-3] DIER, A.F. Tubular joint technology for offshore structures. *Intl. J. of Steel Structures,* December 2005

[A.14.3-4] VAN DE VALK, C.A. Factors Controlling the Static Strength of Tubular T Joints. *Procs. 5th Intl. Conf. on the Behaviour of Offshore Structures,* BOSS 1988, Trondheim, June 1988

[A.14.3-5] American Petroleum Institute, *Proposed API RP2A Upgrade Plan, 1990 - 1999, for Joint Strength and Fatigue Provisions*. API Committee Chaired by N Zettlemoyer, 1990

[A.14.3-6] PECKNOLD, D.A, HA, C.C. and MOHR, W.C. Ultimate Strength of DT Tubular Joints with Chord Preloads, *Proc. 19 Intl. Conf. on Offshore Mechanics and Artic Engineering,* New Orleans, US, February 2000

[A.14.3-7] UK Health and Safety Executive, *Offshore Installations: Guidance on Design, Construction and Certification*, 4th Edition, HMSO, 1990

[A.14.3-8] HOADLEY, P.W. and YURA, J.A. Ultimate Strength of Tubular Joints Subjected to Combined Loads. *Proc. of 17th Offshore Technology Conf.*, OTC 4854, Houston, May 1985

[A.14.3-9] SWENNSON, K.D. and YURA, J.A. *Ultimate Strength of Double-Tee Joints: Interaction Effects*. Phase 3 Final Report, Ferguson Laboratory, U Texas, July 1986

[A.14.4-1] Offshore Research Focus. *Capacity of Overlapping K and YT Joints*. HMSO, **No. 94**, London, April 1993

[A.14.4-2] DEXTER, E.M. and Lee, M.M.K. Effect of Overlap on Strength of K Joints in CHS Tubular Members, *Proc. 6th Intl. Symp. on Tubular Structures*, Melbourne, December 1994

[A.14.4-3] HEALY, B.E. A Numerical Investigation into the Capacity of Overlapped Circular K-Joints, *Procs. 6th Intl. Symp. on Tubular Structures*, Melbourne, December 1994

[A.14.4-4] Bomel Limited, *Design and Reassessment of Tubular Joints for Offshore Structures*, *Chapter 3*, *Static Strength*. 1995

[A.14.4-5] GAZZOLA, F., LEE, M.M.K and DEXTER, E.M. Strength Prediction of Axially Loaded Overlap Tubular K Joints. *Proc. 9th Intl. Offshore and Polar Engineering Conf.*, Brest, France, June 1999

[A.14.4-6] GAZOLLA, F., LEE, M.M.K. and DEXTER, E.M. Strength Sensitivities of Overlap Tubular K — Joints Under Axial Loading, *Proc. 9th Intl. Offshore and Polar Conf.*, Brest, France, 1999

[A.14.5-1] DIER, A.F. and LALANI, M. Guidelines on Strengthening and Repair of Offshore Structures, *Procs. 5th Intl. Conf. on the Behaviour of Offshore Structures* BOSS 1997, **3**, Structures, Delft University, Holland, 1997

[A.14.5-2] TEBBETT, I.E. The Reappraisal of Steel Jacket Structures allowing for Composite Action of Grouted Piles. *Proc. 15th Offshore Technology Conf.,* OTC 4194, Houston, May 1982

[A.14.5-3] Veritec. *Double Skin Grout Reinforced Tubular Joints*, Report No. 84-3564, **1 & 2**, November 1984

[A.14.5-4] TEBBETT, I.E. et al. The Punching Shear Strength of Tubular Joints Reinforced with a Grouted Pile, *Proc. 12th Offshore Technology Conf.,* OTC 3463, Houston, May 1979

[A.14.5-5] LALANI, M. et al. Justification of Enhanced Capacities for As-Welded and Grouted K Joints, *Proc. 18th Offshore Technology Conf.,* OTC 5025, Houston, May 1985

[A.14.5-6] UK HSE, *Grouted and Mechanical Strengthening and Repair of Tubular Steel Structures*, OTH 88 283, HMSO, London, 1988

[A.14.5-7] MSL Engineering Limited, *Development of Grouted Tubular Joint Technology for Offshore Strengthening and Repair — Phase 1 Report*, Doc. Ref. C14100R020, 2, Ascot, England, June 1997

[A.14.6-1] SAWADA, Y. et al, Static and Fatigue Tests on T Joints Stiffened by an Internal Ring, *Proc. 12th Offshore Technology Conf.,* OTC 3422, Houston, May 1979

[A.14.6-2] MURTBY, D.S.R. et al, Structural Efficiency of Internally Ring-Stiffened Steel Tubular Joints, *J. of Structural Engineering*, ASCE, **118**, **No. 11**, November 1992

[A.14.6-3] MARSHALL, P.W. Design of Internally Stiffened Tubular Joints. *Intl. Meeting on Safety Criteria in Design of Tubular Structures*, Tokyo, July 1986

[A.14.6-4] PECKNOLD, D.A., HA, C.C. and MOHR, W.C. Static Strength of Internally Ring-Stiffened DT Joints Subjected to Brace Compression, *Proc. 19th Intl. Conf. on Offshore Mechanics and Artic Engineering*, New Orleans, US, February 2000

[A.14.6-5] ROARK, R.J., YOUNG, W.C. (ed), BUDYNAS, R.G. (ed), *Formulas for Stress and Strain*, McGraw-Hill Education, Seventh Edition, ISBN 007072542X, October, 2001

[A.14.7-1] CHOO, Y.S., VAN DER VEGTE, G.J., ZETTLEMOYER, N., LI, B.H. and LIEW, R.J.Y. Static Strength of Tjoints Reinforced with Doubler or Collar Plates — Part I. Experimental Investigations, *J. of Structural Engineering* – ASCE, 131 (January 2005)

[A.14.7-2] VAN DER VEGTE, G.J., CHOO, Y.S., LIANG, J.X., ZETTLEMOYER, N. and LIEW, R.J.Y. Static Strength of T-joints Reinforced with Doubler or Collar Plates — Part II. Numerical Simulations, *J. of Structural Engineering* – ASCE, 131, (January 2005)

[A.14.7-3] CHOO, Y.S., LIANG, J., VAN DER VEGTE, G.J. and LIEW, R.J.Y. Static Strength of Collar Plate Reinforced CHS X-joints Loaded by In-Plane Bending, *J. of Constructional Steel Research*, **60** (2004)

[A.14.7-4] CHOO, Y.S., LIAN, G.J., VAN DER VEGTE, G.J. and LIEW, R.J.Y. Static Strength of Double Plate Reinforced CHS X-joints Loaded by In-Plane Bending, *J. of Constructional Steel Research*, **60** (2004)

[A.14.8-1] STACEY, A, SHARP, J.V. and NICHOLS, N.W. The Influence of Cracks on the Static Strength of Tubular Joints, *Proc. 15th Intl. Conf. on Offshore Mechanics and Arctic Engineering,* Florence, 1996

[A.14.8-2] STACEY, A, SHARP, J.V. and NICHOLS, N.W. Static Strength of Assessment of Cracked Tubular Joints, *Proc. 15th Intl. Conf. on Offshore Mechanics and Arctic Engineering*, Florence, 1996

[A.14.8-3] HADLEY, I. et al, Static Strength of Cracked Tubular Joints: New Data and Models, *Proc. 17th Intl. Conf. on Offshore Mechanics and Arctic Engineering*, Lisbon, 1998

[A.14.9-1] International Institute of Welding subcommittee XV-E (2003), *Static Design Procedures for Welded Hollow Section Joints — Part 1: Recommendations*, International Institute of Welding, XV-E-03-279, ISO/WD 15-1.2

[A.14.9-2] International Institute of Welding subcommittee XV-E (2003), *Static Design Procedures for Welded Hollow Section Joints — Part 2: Commentary*, International Institute of Welding, XV-E-03-279, ISO/WD 15-1.2

[A.15.1-1] HARWOOD, R.G., BILLINGTON, C.J., BUITRAGO, J., SELE, A.B. and SHARP, J.V. Grouted Pile to Sleeve Connections: Design Provisions for the New ISO Standard for Offshore Structures, *Materials Engineering OMAE Conf.,* **3**, Florence, June 1996

[A.15.2-1] ASME, 2001 Boiler and Pressure Vessel Code Section VIII – Rules for Construction of Pressure Vessels Division 2 – Alternative Rules, *Boiler and Pressure Vessel Code*, ISBN 0791827003, 2001

[A.15.2-2] CHEN, W.C. Fatigue Life Predictions for a Threaded TLP Connector, *Proc. 22nd Offshore Technology Conf*., Paper OTC 5938, Houston, Texas, May 1989

[A.15.2-3] TOPPER, T.H., WETZEL, R.M. and MORROW, J.D. Neuber's Rule Applied to Fatigue of Notched Specimens, *J. of Materials*, **4, No. 1**, March 1969, pp. 200-209

[A.15.2-4] VAN DIEN, J.P., KACZINKI, M.R. and DEXTER, R.J. Fatigue Testing of Anchor Bolts, *ASCE Structures Congress*, **3**, Chicago, 1996

[A.15.2-5] British Standards Institution, *Specification for Bolting for Flanges and Pressure Containing Purposes*, BSI, BS 4882:2001, ISBN: 0 580 17855 2 , November 2001

[A.15.2-6] FRANK, K.H. Fatigue Strength of Anchor Bolts, *J. Structural Division*, ASCE, **106, No. ST6**, June 1980

[A.15.2-7] NICHOLS, N. *Fatigue Performance of Threaded Connections in Air and Sea Water Environments — Derivation of S-N Curves*, *Report*, MATR 0425 report, MaTSU 1997

[A.15.2-8] ASTM, *Standard Specification for Alloy steel and stainless steel bolting materials for high-temperature service,* ASTM A193/A193M-00b, **01.01**, 2003

[A.15.2-9] British Standards Institution, *ISO metric precision hexagon bolts, screws and nuts, Specification,* BSI, BS 3692:2001

[A.15.3-1] Department of Energy, *Grouted and Mechanical Strengthening and Repair of Tubular Steel Offshore Structures*, OTH 88 283, HMSO, London, 1988

[A.15.3-2] MSL Engineering Ltd. *Development of Design Guidance for Neoprene-lined Clamps for Offshore Applications*, Report RR 031, Health & Safety Executive, 2001

[A.16.4-1] VUGTS, J.H., HINES, I.M., NATARAJA, R. and SCHUMM, W. Modal superposition v. direct solution techniques in the dynamic analysis of offshore structures; *Proc. 2nd Intl. Conf. on the Behaviour of Offshore Structures*, Paper 49; ISBN 0-906085-34-9, London, August 1979

[A.16.7-1] NEWLAND, J.E *An introduction to random vibrations and spectral analysis*. Published by Longman, London, 1975 (Second Edition, 1984)

[A.16.7-2] BENDAT, J.S. and PIERSOL, A.G. *Random data: Analysis and measurement procedures*. Wiley-Interscience, 1971 (revised and expanded Second Edition, 1986)

[A.16.7-3] VUGTS, J.H. and KINRA, R.K. Probabilistic Fatigue Analysis of Offshore Structures. *Proc. 9th Offshore Technology Conf.*, OTC 2608, 1976

[A.16.7-4] KINRA, R.K. and MARSHALL, P.W. Fatigue Analysis of the Cognac Platform, *J Petroleum Technology*, SPE 8600, March 1980

[A.16.9-1] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design*, API RP 2A-LRFD. First Edition, 1993

[A.16.10-1] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design,* API RP 2A-LRFD, First Edition, 1993

[A.16.10-2] American Welding Society, *D1.1/D1.1M Structural Welding Code — Steel,* 2004

[A.16.10-3] MARSHALL, P.W. *Design of welded tubular connections — Basis and use of AWS code provisions*, Elsevier Press, Amsterdam, 1992

[A.16.10-4] UK Department of Energy, Background to New Fatigue Design Guidance for Steel Welded Joints in Offshore Structures. *Report of the Den Guidance Notes Revision Drafting Panel*, HMSO, ISBN 0 11 411456 0, 1984

[A.16.10-5] HEALY, B.E. and BUITRAGO, J. Extrapolation procedures for determining SCFs in mid-surface tubular joint models. *6th Intl. Symp. on Tubular Structures*, Monash University, Melbourne, Australia, 1994

[A.16.10-6] EFTHYMIOU, M. Development of SCF formulae and generalised influence functions for use in fatigue analysis. *Recent Developments in Tubular Joint Technology*, OTJ'88, October 1988, London, plus updates

[A.16.10-7] Lloyd's Register of Shipping, *Stress concentration factors for simple tubular joints*. Report OTH 354, Health and Safety Executive, 1997

[A.16.10-8] SMEDLEY, P.A. and FISHER, P. Stress concentration factors for simple tubular joints. *Offshore and Polar Engineering Conf.*, ISOPE-1, Edinburgh, UK, 1991

[A.16.10-9] POTVIN, A.B. et al. Stress concentration in tubular joints. *SPE Journal*, August 1977

[A.16.10-10] WORDSWORTH, A.C. Stress concentration factors at K and KT joints. *Fatigue in Offshore Structural Steels*, ICE, London, 1981

[A.16.10-11] MARSHALL, P.W. and LUYTIES, W.H. Allowable stress for fatigue design. *Proc. 3rd Intl. Conf on the Behaviour of Offshore Structures, BOSS 1982,* Paper S1, **2**, MIT Cambridge, Massachusetts, 1982, pp. 3-25

[A.16.10-12] Edison Welding Institute, *Stress Concentration Factors for Tubular Connections*. EWI Project No. J7266, March 1995

[A.16.10-13] KINRA, R.K. and MARSHALL, P.W. Fatigue Analysis of the Cognac Platform. *J. Petroleum Technology*, Paper SPE 8600, March 1980

[A.16.10-14] MARSHALL, P.W., BUCKNELL, J. and MOHR, W.C. Background to New API fatigue provisions. *Proc. 38th Offshore Technology Conf.* Paper OTC 17295, Houston, May 2005

[A.16.10-15] BUITRAGO, J. et al. Combined Hot-Spot Stress Procedures for Tubular Joints. *Proc. 17th Offshore Technology Conf.* Paper OTC 4775, Houston, May 1984

[A.16.10-16] GIBSTEIN, M. et al. Refined Fatigue Analysis Approach and its Application to the Veslefrikk Jacket. *Third Intl. Symp. on Tubular Structures*, ISTS-3, Finland, 1990

[A.16.10-17] BAERHEIM, M. *Stress Concentrations in Tubular Joints Welded from one Side*. ISOPE Paper No. 96-JSC-285, Los Angeles, May 1996

[A.16.10-18] MSL Engineering Ltd., *Fatigue life implications for design and inspection of single sided welds at tubular joints*. OTO Report 1999 022, UK Health and Safety Executive, June 1999

[A.16.10-19] SMEDLEY, P.A. and FISHER, P.J. Stress concentration factors for ring stiffened tubular joints. *Fourth Intl. symp. on tubular structures*, Delft, 1991

[A.16.10-20] Lloyd's Register of Shipping, *Geometric stress concentration factors for classified details*. Report OTO 97 024, Health and Safety Executive, 1997

[A.16.10-21] Det Norske Veritas, *Fatigue Design of Steel Structures*, Recommended Practice RP C203, Det Norske Veritas, August 2005

[A.16.11-1] Health & Safety Executive, *Background to New Fatigue Guidance for Steel Joints and Connections in Offshore Structures*. HMSO, Report OTH 92 390, December, 1999

[A.16.11-2] Edison Welding Institute, *S-N Curves for Welded Tubular Joints*. Final Report, EWI Project J7267, January 1995

[A.16.11-3] Cranfield University, *Review of the performance of high strength steels used offshore*. Research Report RR105, UK Health and Safety Executive, 2003

[A.16.11-4] Failure Control Engineering and Materials Consultants, *Fatigue design curves for welded joints in air and seawater under variable amplitude loading*, Report OTO 99 058, Health and Safety Executive, 1999

[A.16.11-5] BARKER, J.C. *Data surveys of hydrogen assisted cracking in high strength jack-up steels*, Report OTO 97 072, Health and Safety Executive, 1998

[A.16.11-6] Earl and Wright Consultancy Engineers, *Fatigue Life Assessment of Castings using Fracture Mechanics*. HMSO, Report No. OTH 91 300

[A.16.11-7] MA, A. and SHARP, J.V. Fatigue design of cast steel nodes in offshore structures based on research data. *Proc. Inst. Civil Engineers. Water, Maritime and Energy*, Vol. 124, No. 2, pp. 112-126, Thomas Telford, June 1997

[A.16.12-1] WIRSCHING, P.H. Probabilistic fatigue analysis. *Probabilistic structural mechanics handbook — theory and industrial applications*, Sundararajan C. Raj.(Ed), Chapman and Hall, 1995

[A.16.12-2] MOAN, T. Reliability and risk analysis for design and operations planning of offshore structures, *Proc. ICOSSAR '93*, *6th Int. Conf. on Structural Safety and Reliability*, Innsbruck, Austria, 1994

[A.16.13-1] SOSDIAN, N.R. and CAMPBELL, R.B. Combined tow and in-place platform fatigue design. *Applied Ocean Research*, Vol. 14, Issue 1, Elsevier, 1992

[A.16.14-1] Det Norske Veritas, *Fatigue Design of Steel Structures*. Recommended Practice RP C203, Det Norske Veritas, August 2005

[A.16.15-1] British Standards Institution, BS PD 6493 *Guidance on Methods for Assessing the Acceptability of Flaws in Fusion Welded Structures*, London, 1991

[A.16.15-2] BURDEKIN, F.M. et al. FAULKNER, D.(ed), COWLING, M.J. (ed), INCECIK, A. (ed), and DAS, P.K. (ed), A New Procedure for Assessing Defects in Offshore Structures. *Integrity of Offshore Structures - 5*, Chameleon Press Ltd., London, 1993

[A.16.15-3] HASWELL, J. and HOPKINS, P. A Review of Fracture Mechanics Models for Tubular Joints. *Fatigue and Fracture of Engineering Materials and Structures*, **14**, 1991, pp. 483-497

[A.16.15-4] CONNOLLY, M.P. et al. A Parametric Study of the Ratio of Bending to membrane Stress in Tubular Y and T Joints. *Intl. J. of Fatigue*, **12**, 1990

[A.16.15-5] NEMAN, J.C. and RAJU, I.S. *Stress Intensity Factor Equations for Cracks in Three-Dimensional Finite Bodies Subjected to Tension and Bending Loads*. NASA Technical Memorandum 85793, National Aeronautics and Space Administration, Langley Research Centre, Virginia, April 1984

[A.16.15-6] Lloyd's Register of Shipping, Stress Concentration Factors for Tubular Complex Joints. Complex Joints JIP. *Final Report of Simple Unstiffened Joint SCFs* **No. 3 of 5**, March 1988

[A.16.16-1] MADDOX, S.J., WYLDE, J.G. and YAMAMOTO, N. Significance of weld profile on the fatigue lives of tubular joints. *Proc. Offshore Mechanics and Arctic engineering Conf.* (OMAE), ASME, 1995

[A.16.16-2] Health & Safety Executive, *Background to New Fatigue Guidance for Steel Joints and Connections in Offshore Structures*, Report OTH 92 390, HMSO, December 1999

[A.16.16-3] Edison Welding Institute, *S-N Curves for Welded Tubular Joints*. Final Report, EWI Project J7267, Edison Welding Institute, January 1995

[A.16.16-4] BUITRAGO, J. and ZETTLEMOYER, N. Fatigue of welded joints peened underwater. Complex Joints JIP. *Proc. of 16th Intl. Conf. on Offshore Mechanics and Arctic Engineering*, Materials Engineering volume, Yokohama, Japan, 1997

[A.16.16-5] BUITRAGO, J. and ZETTLEMOYER, N. Quality control of underwater peening. Complex Joints JIP. *Proc. of 16th Intl. Conf. on Offshore Mechanics and Arctic Engineering,* Materials Engineering volume, Yokohama, Japan, 1997

[A.16.16-6] HAAGENSON, P.J. and MADDOX, S.J. *IIW Recommendations for Weld Toe improvements by grinding, TIG dressing and Hammer Peening for steel and aluminium structures*. Document XIII-1815-00, International Institute of Welding, 2000

[A.17.4-1] MURFF, J.D. Pile Capacity in a Softening Soil. *Intl. J. Numerical and Analytical Methods in Geomechanics*, **4**, **No. 2**, April-June 1980, pp. 185-189

[A.17.4-2] RANDOLPH, M.F. Design Considerations for Offshore Piles. In: *Proc. Conf. Geotechnical Practice in Offshore Engineering*, Austin, Texas, New York: American Society of Civil Engineers, April 27-29, 1983, pp. 422-439

[A.17.4-3] TOMLINSON, M.J. *Pile Design and Construction Practice*. Fourth Edition, London: E. and F.N. Spon, 1994

[A.17.4-4] KRAFT, L.M., FOCHT, J.A. and AMARASINGHE, S.F., Friction Capacity of Piles Driven Into Clay. *ASCE J. Geotechnical Engineering Division*, **107**, **No. GT11**, November 1981, pp. 1521-1541

[A.17.4-5] SEMPLE, R.M. and RIGDEN, W.J. Shaft Capacity of Driven Pipe Piles in Clay. In: *Proc. Symp. on Analysis and Design of Pile Foundations*, San Francisco. New York: American Society of Civil Engineers, October 1984

[A.17.4-6] RANDOLPH, M.F. and MURPHY, B.S. Shaft Capacity of Driven Piles in Clay. In: *Proc. 17th Annual Offshore Technology Conf.*, OTC 4883, Houston, Texas, May 1985

[A.17.4-7] PELLETIER, J.H., MURFF, J.D. and YOUNG A.C. Historical Development and Assessment of current API Design Methods for Axially Loaded Piles. *Proc. 27th Annual Offshore Technology Conf.* OTC 7157, Houston, Texas, May 1995

[A.17.4-8] KOLK, H.J. and VAN DER VELDE, E.A Reliable Method to Determine Friction Capacity of Piles Driven into Clays. *Proc. 28th Annual Offshore Technology Conf.*, OTC 7993, May 1996. Houston, Texas, 1996

[A.17.4-9] OLSON, R.E. *Analysis of Pile Response Under Axial Loads.* Report to API, December 1984

[A.17.4-10] CLARKE, J. (Editor). *Large-Scale Pile Tests in Clay*, Thomas Telford, 1993

[A.17.4-11] KRAFT, L.M. and LYONS, C.G. State of the Art: Ultimate Axial Capacity of Grouted Piles, *Proc. 6th Annual Offshore Technology Conf.*, OTC 2081, Houston, Texas, May 1974

[A.17.4-12] O'NEILL, M.W. and HASSAN, K.M. Drilled shafts: Effects of Construction on Performance and Design Criteria. *Proc. Intl. Conf. on Design and Construction of Deep Foundations.* U.S. Federal Highway Administration (FHWA), **1**, 1994, pp. 137-187

[A.17.4-13] LADD, C.C. and FOOTT, R. New Design Procedure for Stability of Soft Clays. *ASCE J. Geotechnical Engineering Division*, vVol. 100, No. GT7, 1974, pp. 763-786

[A.17.4-14] BOGARD, J.D. and MATLOCK, H. Applications of Model Pile Tests to Axial Pile Design. In: *Proc. 22nd Annual Offshore Technology Conf.*, OTC 6376, Houston, Texas, May 1990

[A.17.4-15] LEHANE, B.M. and JARDINE, R.J. Displacement Pile Behavior in Glacial Clay. *Canadian Geotechnical J.*, **31**, **No. 1**, 1994, pp. 79-90

[A.17.4-16] MILLER, T.W., MURFF, J.D. and KRAFT, L.M. Critical State Soil Mechanics Model of Soil Consolidation Stresses Around a Driven Pile. *Proc. 10th Annual Offshore Technology Conf.*, OTC 3307, Houston, Texas, May 1978

[A.17.4-17] RANDOLPH, M.F. and WROTH, C.P. An Analytical Solution for the Consolidation Around a Driven Pile. *Intl. J. for Numerical and Analytical Methods in Geomechanics,* **3**, **No. 3**, July-September 1979, pp. 217-230

[A.17.4-18] BOND, A.J. and JARDINE, R.J. Shaft Capacity of Displacement Piles in a High OCR Clay. *Geotechnique*, **45**, **No. 1**, March 1995, pp. 3-23

[A.17.4-19] O'NEILL, M.W., HAWKINS, R.A. and AUDIBERT, J.M.E. Installation of Pile Group in Overconsolidated Clay. *ASCE J. Geotechnical Engineering Division*, **108**, **GT11**, November 1982, pp. 1369-1386

[A.17.4.20] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Working Stress Design*, API RP 2A-WSD, 21st Edition, 2000, Washington

[A.17.4-21] LEHANE, B.M., SCHNEIDER, J.A. and XU, X. *A Review of Design Methods for Offshore Driven Piles in Siliceous Sand*, University of Western Australia Geomechanics Group, Report No. GEO: 05358, September 2005

[A.17.4-22] LEHANE, B.M., SCHNEIDER, J.A. and XU, X. The UWA-05 Method for Prediction of Axial Capacity of Driven Piles in Sand, in Gourvenec, S. and Cassidy, M. (Eds.), *ISFOG 2005: Proc. 1st Int. Symp. Frontiers in Offshore Geotechnics*, University of Western Australia, Perth, 19-21 September 2005, Taylor & Francis, London, pp. 683-689

[A.17.4-23] KOLK, H.J., BAAIJENS, A.E. and SENDERS, M. Design Criteria for Pipe Piles in Silica Sands, in Gourvenec, S. and Cassidy, M. (Eds.), *ISFOG 2005: Proc. 1st Int. Symp. Frontiers in Offshore Geotechnics*, University of Western Australia, Perth, 19-21 September 2005, Taylor & Francis, London, pp. 711-716

[A.17.4-24] CLAUSEN, C.J.F., AAS, P.M. and KARLSRUD, K. Bearing Capacity of Driven Piles in Sand, the NGI Approach, *ISFOG 2005: Proc. 1st Int. Symp. Frontiers in Offshore Geotechnics*, University of Western Australia, Perth, 19-21 September 2005, Taylor & Francis, London, pp. 677-682

[A.17.4-25] JARDINE, R., CHOW, F., OVERY, R., and STANDING, J. *ICP Design Methods for Driven Piles in Sands and Clays*, Imperial College, Thomas Telford Publishing, London

[A.17.4-26] Fugro, *Axial Pile Capacity Design Method for Offshore Driven Piles in Sand*, Fugro Engineers BV Report No. P1003 to API, Issue 3, 5 August 2004

[A.17.4-27] CUR Centre for Civil Engineering Research and Codes, *Bearing Capacity of Steel Pipe Piles*, CUR, Gouda, CUR-Report, August 2001

[A.17.4-28] RANDOLPH, M.F., LEONG, E.C. and HOULSBY, G.T. One-dimensional Analysis of Soil Plugs in Pipe Piles, *Géotechnique*, Vol. 61, No. 4, pp. 587-598

[A.17.4-29] LEHANE, B.M. and RANDOLPH, M.F. Evaluation of a Minimum Base Resistance for Driven Pipe Piles in Siliceous Sand, *ASCE J. Geotechnical and Geoenvironmental Engineering*, Vol. 128, No. 3, 2002, pp. 198-205

[A.17.4-30] ISO 22476-1, *Geotechnical investigation and testing — Field testing —Part 1: Electrical cone and piezocone penetration tests*

[A.17.4-31] JAMIOLKOWSKI, M., GHIONNA, V.N., LANCELLOTTA, R. and PASQUALINI, E. New Correlations of Penetration Tests for Design Practice, in De Ruiter, J. (Ed.), Penetration Testing 1988: *Proceedings First Intl. Symp. on Penetration Testing*, ISOPT-1, Orlando, 20-24 March 1988, Vol. 1, A.A. Balkema, Rotterdam, pp. 263-296

[A.17.4-32] BALDI, G., BELLOTTI, R., GHIONNA, N., JAMIOLKOWSKI, M. and PASQUALINI, E. Interpretation of CPT's and CPTU's, 2nd Part: Drained Penetration of Sands, in Field Instrumentation and In-Situ Measurements: *Proceedings 4th Intl. Geotechnical Seminar*, 25-27 November 1986, Singapore, Nanyang Technological Institute, Singapore, pp. 143-156

[A.17.4-33] YOUD, T.L., IDRISS, I.M., ANDRUS, R.D., ARANGO, I., CASTRO, G., CHRISTIAN, J.T., DOBRY, R., FINN, W.D.L., HARDER, L.F., HYNES, M.,E., ISHIHARA, K., KOESTER, J.P., LIAO, S.S.C., MARCUSON, W.F., MARTIN, G.R., MITCHELL, J.K., MORIWAKI, Y., POWER, M.S., ROBERTSON, P.K., SEED, R.B. and STOKOE, K.H. Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, *ASCE J. Geotechnical and Geoenvironmental Eng*., Vol. 127, No. 10, October 2001, pp. 817-833

[A.17.4-34] THOMPSON, G.W.L. and JARDINE, R.J., The Applicability of the New Imperial College Design Method to Calcareous Sands. *Proc. Conf. Offshore Site Investigations and Foundation Behaviour*. Society for Underwater Technology, London, 1998, pp. 383-400

[A.17.4-35] KOLK, H.J., Deep Foundations in Calcareous Sediments, in Al-Shafei, K.A. (Ed.), Engineering for Calcareous Sediments: *Proc 2nd Intl. Conf. Eng. for Calcareous Sediments*, Bahrain, 21-24 February 1999, Vol. 2, A.A. Balkema, Rotterdam, 2000, pp. 313-344.

[A.17.4-36] WHITEHOUSE, R., *Scour at Marine Structures*, Thomas Telford Ltd., London, UK, 1998

[A.17.4-37] NEN Nederlands Normalisatie-instituut, *Geotechnics. Calculation Method for Bearing Capacity of Pile Foundation. Compression Piles*, NEN 6743, Nederlandse Normalisatie-Instituut, September 2003

[A.17.4-38] Fugro, Final Report, *Foundation Design — Bridge Piles, Jamuna Bridge*, *Bangladesh*, Fugro Engineers BV, Report No. K-2380/120 to HDEC, 13 June 1995

[A.17.6-1] FOCHT, J.A. and KRAFT, L.M. Axial Performance and Capacity of Piles. In: McClelland, Bramlette and Reifel, Michael D. (editors), *Planning and Design of Fixed Offshore Platforms.* New York, Van Nostrand Reinhold Company, 1986, pp. 763-800

[A.17.6-2] BEA, R.G. Dynamic Response of Marine Foundations. In: *Proc. Ocean Structural Dynamics Symp. '84*, Corvallis Oregon: Oregon State University, 11-13 September 1984

[A.17.6-3] BRIAUD, J.-L., FELIO, G. and TUCKER, L. Influence of Cyclic Loading on Axially Loaded Piles in Clay. Research Report for Phase 2, PRAC 83-42 entitled *Pile Response to Static and Dynamic Loads*. API, December 1984

[A.17.6-4] BRIAUD, J-L. and GARLAND, E. Influence of Loading Rate on Axially Loaded Piles in Clay. Research Report for Phase 1, PRAC 82-42 entitled *Pile Response to Static and Dynamic Loads.* API, March 1984

[A.17.6-5] Panel on Offshore Platforms, Engineering Fixed Offshore Platforms to Resist Earthquakes. *ASCE Specialty Conf. on Earthquake Engineering and Soil Dynamics.* ASCE Geotechnical Engineering Division, June 19-21, 1978

[A.17.6-6] MATLOCK, H. and FOO, S.H.C. Axial Analysis of Piles Using a Hysteretic and Degrading Soil Model. In: *Proc. of Conf. on Numerical Methods in Offshore Piling*, London, Institution of Civil Engineers, 1980, pp. 127-133

[A.17.6-7] POULOS, H.G. Cyclic Axial Pile Response-Alternative Analyses. In: *Proc. Conf. on Geotechnical Practice in Offshore Engineering,* Austin, Texas, New York: American Society of Civil Engineers, 1983, pp. 403-421

[A.17.6-8] KARLSRUD, K., NADIM, F. and HAUGEN, T. Piles in Clay under Cyclic Axial Loading. Field Tests and Computational Modelling. In: *Proc. 3rd Intl. Conf. on Numerical Methods in Offshore Piling,* Nantes, France, Paris: Éditions Technip, 1986, pp. 165-190

[A.17.6-9] BEA, R.G., LITTON, R.W., NOUR-OMID, S., CHANG, J.Y. and VAISH, A.K. A Specialized Design and Research Tool for the Modelling of Near-Field Pile-Soil Interactions. In: *Proc. 16th Annual Offshore Technology Conf.*, OTC 4806, Houston, Texas, May 1984, pp. 249-262

[A.17.6-10] NOVAK, M. and SHARNOUBY, B.E. Stiffness Constants of Single Piles. *ASCE J. of Geotechnical Engineering Division*, **109**, **No. 7**, July 1983, pp. 961-974

[A.17.6-11] ROESSET, J.M. and ANGELIDES, C. Dynamic Stiffness of Piles. In: *Proc. Numerical Methods in Offshore Piling*, London, Institution of Civil Engineers, 1980, pp. 75-81

[A.17.6-12] LYSMER, J. Analytical Procedures in Soil Dynamics. Report No. UCB/EERC-79/29, presented at the *ASCE Geotechnical Engineering Division Specialty Conf. on Earthquake Engineering and Soil Dyna*mics, Pasadena, California, December 1978

[A.17.6-13] MCCLELLAND, B. and EHLERS, C.J. Offshore Geotechnical Site Investigations. In: McClelland, Bramlette and Reifel, Michael D. (editors), *Planning and Design of Fixed Offshore Platforms*. New York, Van Nostrand Reinhold Company, 1986, pp. 224-265

[A.17.6-14] WOOD, D.M. Laboratory Investigations of the Behaviour of Soils under Cyclic Loading: A Review. In: *Soil Mechanics—Transient and Cyclic* Loads, G. N. Pande and O. C. Zienkiewicz (editors), John Wiley & Sons, New York, 1982, Chapter 20

[A.17.6-15] BOGARD, J.D., MATLOCK, H., AUDIBERT, J.M.E. and BAMFORD, S.R. Three Years' Experience with Model Pile Segment Tool Tests. In: *Proc. 17th Annual Offshore Technology Conf.*, OTC 4848, Houston, Texas, 1985

[A.17.6-16] PELLETIER, J.H. and DOYLE, E.H. Tension Capacity in Silty Clays—Beta Pile Test. In: *Proc. 2nd Intl. Conf. on Numerical Methods in Offshore Piling,* Austin, Texas, University of Texas, 1982, pp. 163-181

[A.17.6-17] MCANOY, R.P.L., CASHMAN, A.C. and PURVIS, O. Cyclic Tensile Testing of a Pile in Glacial Till. In: *Proc. 2nd Intl. Conf. on Numerical Methods in Offshore Piling,* Austin, Texas, University of Texas, 1982, pp. 257-291

[A.17.6-18] GALLAGHER, K.A. and ST. JOHN, H.D. Field Scale Model Studies of Piles as Anchorages for Buoyant Platforms. *European Offshore Petroleum Conf.*, Paper EUR 135, London, England, 1980

[A.17.6-19] ARUP, O. et al. *Research on the Behaviour of Piles as Anchors for Buoyant Structures — Summary Report*. Offshore Technology Report OTH 86 215, Department of Energy, London, 1986

[A.17.6-20] PELLETIER, J.H. and SGOUROS, G.E. Shear Transfer Behavior of a 30-inch Pile in Silty Clay. In: *Proc. 19th Annual Offshore Technology Conf.*, OTC 5407. Houston, Texas, April 1987

[A.17.6-21] BEA, R.G., VAHDANI, S., GUTTMAN, S.I., MEITH, R.M. and PAULSON, S.F. Analysis of the Performance of Piles in Silica Sands and Carbonate Formations. In: *Proc. 18th Annual Offshore Technology Conf.*, OTC 5145, Houston, Texas, May 1986

[A.17.7-1] KRAFT, L.M., FOCHT, J.A. and AMARASINGHE, S.F. Friction Capacity of Piles Driven Into Clay. *ASCE J. Geotechnical Engineering Division*, **107**, **No. GT11**, November 1981, pp. 1521-1541

[A.17.7-2] COYLE, H.M. and REESE, L.C. Load Transfer for Axially Loaded Piles in Clay. *ASCE J. Soil Mechanics and Foundations Division*, **92**, **No. SM2**, March 1966, pp. 1-26

[A.17.8-1] MATLOCK, H. Correlations for Design of Laterally Loaded Piles in Soft Clay. In: *Proc. 2nd Annual Offshore Technology Conf.*, OTC 1204, Houston, Texas, April 1970

[A.17.8-2] REESE, L.C. and COX, W.R. Field Testing and Analysis of Laterally Loaded Piles in Stiff Clay. In: *Proc. 5th Annual Offshore Technology Conf.*, OTC 2312, Houston, Texas, April 1975

[A.17.8-3] O'NEILL, M.W. and MURCHISON, J.M. *An Evaluation of p-y Relationships in Sands*, prepared for the American Petroleum Institute Report PRAC 82-41-1. Houston, University of Houston, March 1983

[A.17.8.4] GEORGIADIS, M. Development of p-y Curves for Layered Soils. *Proc. Conf.on Geotechnical Practice in Offshore Engineering*, Austin, Texas, New York: American Society of Civil Engineers, 1983, pp. 536-545

[A.17.8-5] WHITEHOUSE, R., *Scour at Marine Structures*, Thomas Telford Ltd., London, UK, 1998

[A.17.9-1] POULOS, H.G. and RANDOLPH, M.F. Pile Group Analysis: A Study of Two Methods. *ASCE J. Geotechnical Engineering Division*, **109**, **No. 3**, pp. 335-372

[A.17.9-2] FOCHT, J.A. and KOCH, K.J. Rational Analysis of the Lateral Performance of Offshore Pile Groups. In: *Proc. 5th Annual Offshore Technology Conf.*, OTC 1896, Houston, Texas, April/May 1973

[A.17.9-3] O'NEILL, M.W. Group Action in Offshore Piles. In: *Proc. Conf. on Geotechnical Practice in Offshore Engineering*, Austin, Texas, New York: American Society of Civil Engineers, 1983, pp. 25-64

[A.17.9-4] DIGRE, K.A, BRASTED, L.K. and MARSHALL, P.W. The Design of the Bullwinkle Platform. In: *Proc. 21st Annual Offshore Technology Conf.*, OTC 6050, Houston, Texas, 1989

[A.17.10-1] CHEN, W.F. and LUI, E.M. Column with End Restraints and Bending in Load and Resistance Factor Design. T. R. Higgins' lectureship Award Winning Paper, *AISC Engineering J.*, Third Quarter 1985, pp. 105-132

[A.18.4-1] ISO 8501-1:2007, *Preparation of steel substrates before application of paints and related products — Visual assessment of surface cleanliness — Part 1: Rust grades and preparation grades of uncoated steel substrates and of steel substrates after overall removal of previous coatings*

[A.18.4-2] ISO 8503:1988 (all parts), *Preparation of steel substrates before application of paints and related products — Surface roughness characteristics of blast-cleaned steel substrates*

[A.18.4-3] ISO 12944-5:1998, *Paints and varnishes — Corrosion protection of steel structures by protective paint systems — Part 5: Protective paint systems*

[A.18.4-4] NORSOK Standard M-CR-501: *Surface Preparation and Protective Coating*, Oijeindustriens Lansforening, Norway, Revision 6, 2004

[A.18.4-5] CEN EN 12473:2000, *General Principles of Cathodic Protection in Sea Water*

[A.18.4-6] CEN EN 12495:2000, *Cathodic Protection for Fixed Steel Offshore Structures*

[A.18.4-7] NACE Standard RP0176-2003, *Corrosion control of fixed offshore steel structures associated with petroleum production*, National Association of Corrosion Engineers, 2003

[A.18.4-8] NACE Standard RP0387-99, *Metallurgical and Inspection Requirements for Cast Sacrificial Anodes for Offshore Applications*, National Association of Corrosion Engineers, 1999

[A.18.4-9] BS 7361-1:1991, *Cathodic Protection. Code of Practice for Land and Marine Applications*

[A.18.4-10] DNV RP B401 (2005), *Cathodic Protection Design*, Det Norske Veritas, Norway, 2005

[A.18.4-11] NORSOK Standard M-CR-503: *Cathodic Protection*, Oljeindustriens Landsforening, Norway, Revision 2, 1999

[A.18.4-12] Publication 90/102: *Design and Operational Guidance on Cathodic Protection of Offshore Structures, Subsea Installations and Pipelines*, The Marine Technology Directorate Ltd, UK, 1990

[A.19.2-1] Norwegian Technology Centre, *Material Data Sheets for Structural Steel*, NORSOK M-CR-120, **1**, December 1994

[A.19.5-1] Engineering Equipment and Materials Users Association, *Specification for Structural Castings for Use Offshore*, Publication 176, EEMUA

[A.19.6-1] *The Strength of Grouted Pile-Sleeve Connections*, Offshore Technology Report OTH 86 210

[A.19.6-2] American Society for Testing and Materials, *Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)*, ANSI/ASTM C109/109M-2, 2002

[A.20.2-1] Engineering Equipment and Materials Users Association, *Construction Specification for Fixed Offshore Structures in the North Sea*, Publication 158, EEMUA

[A.20.2-2] American Welding Society, *American Welding Society's D1.1/D1.1M Structural Welding Code — Steel*, 2004

[A.20.2-3] Norwegian Technology Centre, *Materials selection*, NORSOK M-001, **1**, NTS, 2002

[A.20.2-4] CSA W59, *Welded Steel Construction (Metal Arc Welding)*, Canadian Standards Association, 2003

[A.20.2-5] American Petroleum Institute, *Preproduction Qualification for Steel Plates for Offshore Structures,* API RP2Z, Fourth Edition, 2005

[A.20.2-6] American Petroleum Institute, *Steel Plates for Offshore Structures, Produced by Thermo-Mechanical Control Processing (TMCP),* API Spec 2W, Fourth Edition, 1999

[A.20.2-7] American Petroleum Institute, *Steel Plates, Quenched-and-tempered, for Offshore Structures,* API Spec 2Y, Fourth Edition, 1999

[A.20.2-8] American Society for Testing and Materials, *Standard Test Method for Conducting Drop-Weight Test to Determine Nil-Ductility Transition Temperature of Ferritic Steels*, ASTM E208-95a, 2002

[A.20.2-9] American Petroleum Institute, *Carbon Manganese Steel Plate for Offshore Platform Tubular Joints,* API Spec 2H, Eighth Edition, 1999

[A.20.2-10] European Committee for Standardization, *Weldable structural steels for fixed offshore structures — Technical delivery conditions*, EN 10225, 2001

[A.21.2-1] American Petroleum Institute, *Specification for Quality Programs for the Petroleum and Natural Gas Industry,* ANSI/API Spec Q1, 1999

[A.21.3-1] American Welding Society, *Standard for AWS Certification of Welding Inspectors*, AWS QC-1:2001

[A.21.3-2] Canadian Standards Association, *Certification of Welding Inspectors,* CSA Standard W178.2

[A.21.3-3] http://www.cswip.com, TWI Cerification Ltd

[A.21.3-4] http://www.twi.co.uk/j32k/unprotected/band_1/bgas.html, TWI Cerification Ltd

[A.21.3-5] www.bindt.org, British Institute of Non-Destructive Testing

[A.21.3-6] American Society for Nondestructive Testing, *Central certification program*, ASNT CP-1, 2005

[A.21.3-7] American Society for Nondestructive Testing, *Recommended practice for qualification and certification of NDE personnel*, ASNT SNT-TC-1A, 2001

[A.21.3-8] ISO 9712, *Non-destructive testing — Qualification and certification of personnel*

[A.21.3-9] American Petroleum Institute, *Recommended practice for ultrasonic and magnetic examination of offshore structural fabrication and guidelines for qualification of technicians,* API RP2X

[A.22.5-1] SMITH, E.A.L. Pile Driving Analysis by the Wave Equation, *J. of Soil Mechanics and Foundation Design,* ASCE, **86**, August, 1960

[A.22.5-2] GIBSON, G. and COYTE, H.M. *Soil Damping Constants Related to Common Soil Properties in Sand and Clays,* Texas Transportation Institute, Report no. 125-1, Texas A&M University, 1968

[A.22.5-3] TOOLAN, F.E. and FOX, D.A. Geotechnical Planning of Piled Foundations for Offshore Platforms, *Proc. Institutions of Civil Engineers,* Part 1, 1977, pp. 221-224

[A.22.5-4] HEEREMA, E.P. Relationship between Wall Friction, Displacement and Horizontal Stress in Clay and Sand, for Pile Drivability Analysis, *Ground Engineering*, January 1979

[A.22.5-5] ROUSSEL, H.J. *Pile Driving Analysis of Large Diameter High Capacity Offshore Pipe Piles*, (PhD Thesis), Tulane University, March 1979

[A.22.5-6] HEEREMA, E.P. Dynamic Point Resistance in Sand and Clay for Pile Drivability Analysis, *Ground Engineering*, 1981

[A.22.5-7] SEMPLE, R.S. and GEMEINHARDT, J.P. Stress History Approach to Analysis of Soil Resistance to Pile Driving, *Proc. 14th Offshore Technology Conf.*, Houston, Texas, Paper OTC 3869, May 1981

[A.22.5-8] STEVENS, R.S., WILTSIE, E.A. and TURTON, T.H. Evaluating Pile Drivability for Hard Clay, Very Dense Sand and Rock, *Proc. 15th Offshore Technology Conf.*, Houston, Texas, Paper OTC 4205, May 1982

[A.22.5-9] ZANDWIJK, C.J., VOETEN, J.A.J. and HEEREMA, E.P. An Improved Pile Drivability Theory for Gulf of Mexico Soils, *Proc. 16th Offshore Technology Conf.*, Houston, Texas, Paper OTC 4503, May 1983

[A.22.5-10] DOLWIN, J., KHORSHID, M.S. and VAN GOUDOEVER, P. Evaluation of driven pile capacity – Methods and results, Engineering for Calcareous Sediments, *Proc. Intl. Conf. on Calcareous Sediments*, **2**, Perth, 15-18 March 1988, pp. 409-428

[A.22.5-11] HUSSEIN, M., RAUSCHE, F. and LIKINS, G. Bearing capacity of piles and conductors by dynamic analyses, Engineering for Calcareous Sediments, *Proc. Intl. Conf. on Calcareous Sediments*, **2**, Perth, 15-18 March 1988, pp. 439-447

[A.22.5-12] PUECH, A., POULET, D. and BOISARD, P.A. *Procedure to Evaluate Pile Drivability Analysis in the Difficult Soil Conditions of the Southern Part of the Gulf of Guinea*, OTC Paper No.8237, Houston, May 1990

[A.22.5-13] RANDOLPH, M.F. Analysis of Stress-Wave Data from Pile tests at Pentre and Tilbrook, *Proc. Conf. on Large-Scale Pile Tests in Clay*, London, June, 1992, pp. 436-453

[A.22.5-14] BRUCY, F. and MEUNIER, J. Pile Resistance at a Dense Sand Site, *Proc. Intl. Conf. on Application of Stress Wave Theory to Piles*, The Hague, September 1992, pp. 69-76

[A.22.5-15] DANINGER, B.R., PACHECO, M.P., COSTA, A.M. and LOPES, F.R. Back-Analysis of Closed-Ended Pipe Piles for an Offshore Platform, *Proc. Intl. Conf. Application of Stress-Wave Theory to Piles*, The Hague, September 1992, pp. 557-562

[A.22.5-16] MELLO, J.R.C. and GALGOUL, N.S. Piling and Monitoring of Large Diameter Closed-Toe Pipe Piles, *Proc. Intl. Conf. on Application of Stress Wave Theory to Piles*, The Hague, September 1992, pp. 443-448

[A.22.5-17] COLLIAT, J.L., VERGOBBI, P. and PUECH, A. *Friction, Degradation and Set-Up in Hard Clays Offshore Congo and Angola*, OTC Paper No. 7192, Houston, May 1993

[A.22.5-18] DOYLE, E.H. Pile Installation Performance for Four TLP's in the Gulf of Mexico, *Proc. 31st Annual Offshore Technology Conf.,* OTC Paper No. 10826, Houston, Texas, May 1999

[A.22.5-19] STEVENS, R.F., WILTSIE, E.A. and MIDDLEBROOKS, B.S. Controlled Hard Driving. *Proc. 2nd Intl. Conf. on Application of Stress-Wave Theory on Piles*, Stockholm, May 1984

[A.22.5-20] WILTSIE, E.A., STEVENS, R.F. and VINES, W.R. Pile Installation Acceptance in Strong Soils. *Proc. 2nd Intl. Conf. on Application of Stress-Wave Theory on Piles*, Stockholm, May 1984

[A.23.3-1] KALLABLY, J. and O'CONNOR, P.E. (1994) An integrated approach to underwater survey and damage assessment of offshore platforms. OTC 7487, Volume IV, 26th *Annual Offshore Technology Conf.*, Houston, Texas, 2-5 May, 1994, pp. 189-202

[A.23.4-1] API, *Recommended Practice for Ultrasonic and Magnetic Examination of Offshore Structural Fabrication for Qualification of Technicians*, API RP2X. Third Edition, November 1996

[A.23.8-1] CSWIP (1990). *Requirements for the certification of underwater (diver) inspectors, ROV inspectors and underwater inspection controllers*. Document No CSWIP-DIV-7-95-Part 1 (TWI Certification, Abington, Cambridge, UK)

[A.23.8-2] European Committee for Standardization (CEN), EN 473, *Non destructive testing — Qualification and certification of NDT personnel — General principles*, 2000

[A.24.5-1] DNV/SINTEF/BOMEL *Ultiguide – Best Practice guidance for use of non-linear analysis methods in documentation of ultimate limit states for jacket type offshore structure*, DNV, April 1999

[A.24.5-2] American Petroleum Institute (1993), *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Working Stress Design*, API RP 2A-WSD, 21st Edition

[A.24.5-3] DIGRE, K.A., KRIEGER, W. F., WISCH, D.J. and PETRAUSKAS, C. API RP 2A Draft Section 17 Assessment of Existing Platforms, *Procs. 7th Intl. Conf. on the Behaviour of Offshore Structures*, BOSS 1994, Cambridge, Massachusetts, 1994

[A.24.5-4] KALLABY, J. and O'CONNOR, P. An Integrated Approach for Underwater Survey Damage Assessment of Offshore Platforms, *Proc. 26th Offshore Technology Conf.*, OTC Paper 7487, Houston, May 1994

[A.24.5-5] KALLABY, J., LEE, G., CRAWFORD, C., LIGHT, L., DOLAN, D. and CHEN, J.H. Structural Assessment of Existing Platforms, *Proc. 26th Offshore Technology Conf.*, OTC Paper 7483, Houston, May 1994

[A.24.5-6] KRIEGER, W.F., BANON, H., LLOYD, J.R., DE, R.S., DIGRE, K. A., NAIR, D., IRICK, J.T. and GUYNES, S. Process for Assessment of Existing Platforms to Determine Fitness for Purpose, *Proc. 26th Offshore Technology Conf.*, OTC Paper 7482, Houston, May 1994

[A.24.5-7] PUSKAR, F.J., AGGARWAL, R.K., CORNELL, C.A., MOSES, F. and PETRAUSKAS, C. A Comparison of Analytically Predicted Platform Damage to actual Platform Damage During Hurricane Andrew, *Proc. 26th Offshore Technology Conf.*, OTC Paper 7473, Houston, May 1994

[A.24.5-8] PETRAUSKAS, C., FINNEGAN, T.D., HEIDEMAN, J., SANTALA, M., VOGEL, M. and BEREK, G., Metocean Criteria/Loads for Use In Assessment of Existing Offshore Platforms, *Proc. 26th Offshore Technology Conf.*, OTC Paper 7484, Houston, May 1994

[A.24.5-9] CRAIG, M.J.K. and DIGRE, K.A. Assessments of High Consequence Platforms — Issues and Applications, *Proc. 26th Offshore Technology Conf.*, OTC Paper 7485, Houston, May 1994

[A.24.9-1] DE, R.S. Risk Analysis Methodology for Developing Design Assessment Criteria for Fixed Offshore Structures, *Proc. 27th Offshore Technology Conf.*, OTC Paper 7755, Houston, May 1995

[A.24.9-2] VAN DE GRAAF, J.W., TROMANS, P.S. and EFTHYMIOU, M. The Reliability of Offshore Structures and its Dependence on Design Code and Environment, *Proc. 27th Offshore Technology Conf.*, Paper 7382, Houston, May 1994

[A.24.9-3] EFTHYMIOU, M., VAN DE GRAAF, J.W., TROMANS, P.S. and HINES, I.M. Reliability Based Criteria for Fixed Steel Offshore Platforms, *Proc. Conf. OMAE-96*, Florence, Italy, June 1996

[A.24.10-1] TURNER, J.W., WISCH, D.J. and GUYNES, S.A Review of Operations and Mitigation Methods for Offshore Platforms, *Proc. 26th Offshore Technology Conf.*, OTC Paper 7486, Houston, May 1994

[B.1] British Standards Institution (BSI), *Fracture mechanics toughness tests*, BS 7448, Parts 1, 2 and 4, London, 1991, 1997 and 1997

[B.2] ASTM International, *Standard Test Method for Crack-Tip Opening Displacement (CTOD) Fracture Toughness Measurement,* E1290-02, 2002

[C.1] American Society for Testing and Materials, *Standard Specification for Carbon Structural Steel*, ANSI/ASTM A36/A36M-03, 2003

[C.2] American Society for Testing and Materials, *Standard Specification for Structural Steel for Ships*, ANSI/ASTM A131/A131M-01, 2001

[C.3] American Society for Testing and Materials, *Standard Specification for Pressure Vessel Plates, Carbon Steel, for Moderate- and Lower-Temperature Service*, ANSI/ASTM A516/A516M-03, 2003

[C.4] American Society for Testing and Materials, *Standard Specification for Structural Carbon Steel Plates of Improved Toughness*, ANSI/ASTM A573/A573M-00a, 2000

[C.5] American Society for Testing and Materials, *Standard Specification for Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates, and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges*, ANSI/ASTM A709/A709M-03a, 2003

[C.6] American Society for Testing and Materials, *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*, ANSI/ASTM A572/A572M-03ae1, 2003

[C.7] American Petroleum Institute, *Carbon Manganese Steel Plate with Improved Toughness for Offshore Structures*, API Spec 2MT1, 2001

[C.8] American Petroleum Institute, *Carbon Manganese Steel Plate for Offshore Platform Tubular Joints*, API Spec 2H, 1999

[C.9] American Petroleum Institute, *Steel Plates for Offshore Structures, Produced by Thermo-Mechanical Control Processing (TMCP)*, API Spec 2W, 1999

[C.10] American Petroleum Institute, *Steel Plates, Quenched-and-tempered, for Offshore Structures*, API Spec 2Y, 1999

[C.11] American Society for Testing and Materials, *Standard Specification for Pressure Vessel Plates, Heat-Treated, Carbon-Manganese-Silicon Steel*, ANSI/ASTM A537/A537M-95, 1995

[C.12] American Society for Testing and Materials, *Standard Specification for Normalized High-Strength Low-Alloy Structural Steel Plates*, ANSI/ASTM A633/A633M-01, 2001

[C.13] American Society for Testing and Materials, *Standard Specification for Quenched-and-Tempered Carbon and High-Strength Low-Alloy Structural Steel Plates*, ANSI/ASTM A678/A678M-00a, 2000

[C.14] American Society for Testing and Materials, *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*, ANSI/ASTM A6/A6M-06, 2006

[C.15] American Society for Testing and Materials, *Standard Specification for General Requirements for Steel Plates for Pressure Vessels*, ANSI/ASTM A20/A20M-06, 2006

[C.16] American Society for Testing and Materials, *Standard Specification for Straight-Beam Ultrasonic Examination of Plain and Clad Steel Plates for Special Applications*, ANSI/ASTM A578/A578M-96, 2001

[C.17] American Society for Testing and Materials, *Standard Specification for Structural Steel Shapes*, ANSI/ASTM A992/A992M-06a, 2006

[C.18] American Petroleum Institute, *Rolled Shapes with Improved Notch Toughness*, API Spec 2MT2, 2002

[C.19] American Petroleum Institute, *Line Pipe*, API Spec 5L, 1999

[C.20] American Society for Testing and Materials, *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*, ANSI/ASTM A53/A53M-02, 2002

[C.21] American Society for Testing and Materials, *Standard Specification for Seamless Carbon Steel Pipe for High-Temperature Service*, ANSI/ASTM A106/A106M-02a, 2002

[C.22] American Society for Testing and Materials, *Standard Specification for Electric-Resistance-Welded Steel Pipe*, ANSI/ASTM A135/A135M-01, 2001

[C.23] American Society for Testing and Materials, *Standard Specification for Electric-Fusion (Arc)-Welded Steel Pipe (NPS 4 and Over)*, ANSI/ASTM A139/A139M-00, 2000

[C.24] American Society for Testing and Materials, *Standard Specification for Seamless and Welded Steel Pipe for Low-Temperature Service*, ANSI/ASTM A333/A333M-04, 2004

[C.25] American Society for Testing and Materials, *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*, ANSI/ASTM A500/A500M-03a, 2003

[C.26] American Petroleum Institute, *Fabrication of Structural Steel Pipe*, API Spec 2B, 2001

[D.1] European Committee for Standardization, *Hot rolled products of structural steels,* EN 10025, 2004

[D.2] European Committee for Standardization, *Weldable structural steels for fixed offshore structures — Technical delivery conditions*, EN 10225, 2001

[D.3] NORSOK M-120, *Material data sheets for structural steel*, **1** Norwegian Technology Centre, NTS, 2000

[D.4] European Committee for Standardization, *Hot finished structural hollow sections of non-alloy and fine grain steels*, EN 10210, 2006

[D.5] European Committee for Standardization, *Cold formed welded structural hollow sections of non-alloy and fine grain steels*, EN 10219, 2006

[H.1] *Canada Oil and Gas Operations Act*, http://laws.justice.gc.ca

[H.2] *Canada Petroleum Resources Act*, http://laws.justice.gc.ca

[H.3] *Canada-Nova Scotia Offshore Petroleum Resources Accord Implementation Act*, http://laws.justice.gc.ca

[H.4] *Canada-Newfoundland Atlantic Accord Implementation Act*, http://laws.justice.gc.ca

[H.5] *Canada-Nova Scotia Offshore Petroleum Resources Accord Implementation (Nova Scotia) Act*, http://www.gov.ns.ca/just/regulations

[H.6] *Canada-Newfoundland and Labrador Atlantic Accord Implementation Newfoundland and Labrador Act*, http://www.hoa.gov.nl.ca

[H.7] CSA S471-04, *General requirements, design criteria, the environment, and loads*, Canadian Standards Authority, 2004

[H.8] Det Norske Veritas, *Buckling Strength of Plated Structures,* Recommended Practice DNV-RP-C201, DNV, October, 2002

[H.9] American Petroleum Institute, *Design of Flat Plate Structures*, API Bulletin 2V, 2000

[H.10] CSA W59-03, *Welded Steel Construction (Metal Arc Welding)*, Canadian Standards Authority, 2003

[H.11] CSA-S473-04, *Steel structures*, Canadian Standards Authority, 2004

[H.12] CSA W47.1-03, *Certification of Companies for Fusion Welding of Steel Structures*, Canadian Standards Authority, 2003

[H.13] CSA W178.1-02, *Certification of Welding Inspection Organizations*, Canadian Standards Authority, 2002

[H.14] CSA W178.2-01, *Certification of Welding Inspectors*, Canadian Standards Authority, 2001

ICS 75.180.10

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