DNV·GL

OFFSHORE STANDARDS

DNVGL-OS-C101 Edition July 2018

Design of offshore steel structures, general - LRFD method

DNV GL AS

FOREWORD

DNV GL offshore standards contain technical requirements, principles and acceptance criteria related to classification of offshore units.

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

CHANGES – CURRENT

This document supersedes the July 2017 edition of DNVGL-OS-C101.

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

Changes July 2018

Topic	*Reference*	*Description*
Minor update	Ch.1 Sec.1 Table 1	Include DNVGL-RP-C212 and DNVGL-CG-0129 as reference document.
	Ch.1 Sec.1 Table 5 and Ch.1 Sec.1 Table 6	Update definition and abbreviation of lowest mean daily average temperature (LMDAT).
	Ch.2 Sec.3 [3.3.5]	Updated clause by including requirements on fabrication and tolerances for fatigue critical details.
FLS	Ch.2 Sec.5 [1.2]	Updated subsection on design fatigue factors (DFF) including restructured table.
ALS	Ch.2 Sec.6	Section on accidental limit states completely updated and restructured, brought in line with the general principles in DNVGL-OS-A101. Accidental events removed, referring to DNVGL-OS-A101 instead.
Update based on output from NORMOOR JIP	Ch.2 Sec.10 Table 1 and Ch.2 Sec.10 Table 2	Updated tables with safety factors to be applied in soil design reflecting update of mooring system safety factors in DNVGL-OS-E301.

Editorial corrections

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

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CHAPTER 1 INTRODUCTION

SECTION 1 INTRODUCTION

1 General

1.1 Introduction

1.1.1 This offshore standard provides principles, technical requirements and guidance for the structural design of offshore structures.

1.1.2 DNVGL-OS-C101 is the general part of the DNV GL offshore standards for structures. The design principles and overall requirements are defined in this standard. The standard is primarily intended to be used in design of a structure where a supporting object standard exists, but may also be used as a standalone document for objects where no object standard exist.

1.1.3 When designing a unit where an object standard exists, the object standard (DNVGL-OS-C10x) for the specific type of unit shall be applied. The object standard gives references to this standard when appropriate.

1.1.4 In case of deviating requirements between this standard and the object standard, requirements of this standard shall be overruled by specific requirements given in the object standard.

1.2 Objectives

The objectives of this standard are to:

— provide an internationally acceptable level of safety by defining minimum requirements for structures and structural components (in combination with referred standards, recommended practices, guidelines, etc.)
— serve as a contractual reference document between suppliers and purchasers
— serve as a guideline for designers, suppliers, purchasers and regulators.

1.3 Scope and application

1.3.1 The standard is applicable to all types of offshore structures of steel.

1.3.2 For other materials, the general design principles given in this standard may be used together with relevant standards, codes or specifications.

1.3.3 The standard is applicable to the design of a unit's complete structures including hull structure, substructures, topside structures, and foundations.

1.3.4 This standard gives requirements for the following:

— design principles
— structural categorisation
— material selection and inspection principles
— design loads
— load effect analyses
— design of steel structures and connections
— corrosion protection

— foundation design.

1.3.5 For application of this standard as technical basis for classification, see Ch.3.

1.3.6 Flag and shelf state requirements are not covered by this standard.

Guidance note:

Governmental regulations may include requirements in excess of the provisions of this standard depending on the type, location and intended service of the offshore unit or installation. The 100 year return period is used to ensure harmonisation with typical Shelf State requirements and the code for the construction and equipment of mobile offshore drilling units (MODU code).

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

1.4 Use of other codes and standards

1.4.1 In case of conflict between the requirements given in this standard and a reference document other than DNV GL documents, the requirements of this standard shall prevail.

1.4.2 Where reference is made to codes other than DNV GL documents, the latest revision of the documents shall be applied, unless otherwise specified.

1.4.3 When checks are performed according to other than DNV GL codes/standards, the load and material factors as given in this standard shall be applied.

2 References

2.1 General

2.1.1 The DNV GL documents in Table 1 and Table 2 and recognised codes and standards in Table 3 are referred to in this standard.

2.1.2 The latest revision in force of the DNV GL reference documents in Table 1 and Table 2 applies. These include acceptable methods for fulfilling the requirements in this standard. See also current DNV GL list of publications.

2.1.3 When designing a unit where an object standard exists, the object standard for the specific type of unit shall be applied, see Table 2. The object standard gives references to this standard when appropriate, see also [1.1.3] and [1.1.4].

2.1.4 Other recognised codes or standards may be applied provided it is shown that they meet or exceed the level of safety of the actual relevant DNV GL offshore standard. Use of other standards/codes shall be agreed in advance, unless specifically referred to in this standard.

Table 1 DNV GL and DNV reference documents

Document code	*Title*
DNV-CN-30.6	Structural reliability analysis of marine structures
DNVGL-CG-0128	Buckling
DNVGL-CG-0129	Fatigue assessment of ship structures

Document code	*Title*
DNVGL-OS-A101	Safety principles and arrangement
DNVGL-OS-B101	Metallic materials
DNVGL-OS-C401	Fabrication and testing of offshore structures
DNVGL-OS-E301	Position mooring
DNVGL-ST-N001	Marine operations and marine warranty
DNVGL-RU-SHIP Pt.1 Ch.3	Documentation and certification types, general
DNVGL-ST-0378	Standard for offshore and platform lifting appliances
DNVGL-RP-B401	Cathodic protection design
DNVGL-RP-C201	Buckling strength of plated structures
DNVGL-RP-C202	Buckling strength of shells
DNVGL-RP-C203	Fatigue design of offshore steel structures
DNVGL-RP-C204	Design against accidental loads
DNVGL-RP-C205	Environmental conditions and environmental loads
DNVGL-RP-C208	Determination of structural capacity by non-linear finite element analysis methods
DNVGL-RP-E301	Design and installation of fluke anchors
DNVGL-RP-C212	Offshore soil mechanics and geotechnical engineering
DNVGL-RP-E302	Design and installation of plate anchors in clay
DNVGL-RP-E303	Geotechnical design and installation of suction anchors in clay

Table 2 DNV GL offshore object standards for structural design

Document code	*Title*
DNVGL-OS-C102	Structural design of offshore ships-shaped units
DNVGL-OS-C103	Structural design of column-stabilised units - LRFD method
DNVGL-OS-C104	Structural design of self-elevating units - LRFD method
DNVGL-OS-C105	Structural design of TLP - LRFD method
DNVGL-OS-C106	Structural design of deep draught floating units

Table 3 Other references

Document code	*Title*
AISC	AISC Steel construction manual
API RP 2A	Planning, designing, and constructing fixed offshore platforms
EN 1993-1 series	Eurocode 3: Design of steel structures
NACE TPC 3	Microbiologically Influenced Corrosion and Biofouling in Oilfield Equipment

Document code	Title
International life-saving appliances (LSA) code	1996 and amended in 2006 (adopted by the Maritime Safety Committee of the Organization by resolution MSC.48(66), as amended)
NORSOK N-003	Actions and action effects
NORSOK N-004	Design of steel structures
ISO 19902	Petroleum and natural gas industries, fixed steel offshore structures

3 Definitions

3.1 Verbal forms

Table 4 Definition of verbal forms

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

3.2 Terms

Table 5 Definition of terms

Term	Definition
accidental limit states	ensures that the structure resists accidental loads and maintain integrity and performance of the structure due to local damage or flooding
atmospheric zone	the external surfaces of the unit above the splash zone
cathodic protection	a technique to prevent corrosion of a steel surface by making the surface to be the cathode of an electrochemical cell
characteristic load	the reference value of a load to be used in the determination of load effects. The characteristic load is normally based upon a defined fractile in the upper end of the distribution function for load.
characteristic resistance	the reference value of structural strength to be used in the determination of the design strength. The characteristic resistance is normally based upon a 5% fractile in the lower end of the distribution function for resistance.
characteristic material strength	the nominal value of material strength to be used in the determination of the design resistance. The characteristic material strength is normally based upon a 5% fractile in the lower end of the distribution function for material strength.
characteristic value	the representative value associated with a prescribed probability of not being unfavourably exceeded during the applicable reference period

Term	*Definition*
classification note	the classification notes cover proven technology and solutions which is found to represent good practice by DNV GL, and which represent one alternative for satisfying the requirements stipulated in the DNV GL Rules or other codes and standards cited by DNV GL. The classification notes will in the same manner be applicable for fulfilling the requirements in the DNV GL offshore standards.
coating	metallic, inorganic or organic material applied to steel surfaces for prevention of corrosion
corrosion allowance	extra wall thickness added during design to compensate for any anticipated reduction in thickness during the operation
design brief	an agreed document where owners requirements in excess of this standard should be given
design life	the defined period the unit is expected to operate
design fatigue life	design life × design fatigue factor
design temperature	the design temperature for a unit is the reference temperature for assessing areas where the unit can be transported, installed and operated. The design temperature shall be lower or equal to the lowest mean daily average temperature in air for the relevant areas. For seasonal restricted operations the lowest mean daily average temperature in air for the season may be applied.
design value	the value to be used in the deterministic design procedure, i.e. characteristic value modified by the resistance factor or load factor
driving voltage	the difference between closed circuit anode potential and the protection potential
expected loads and response history	expected load and response history for a specified time period, taking into account the number of load cycles and the resulting load levels and response for each cycle
expected value	the most probable value of a load during a specified time period
fatigue	degradation of the material caused by cyclic loading
fatigue critical	structure with calculated fatigue life near the design fatigue life
fatigue limit states	related to the possibility of failure due to the effect of cyclic loading
foundation	a device transferring loads from a heavy or loaded object to the vessel structure
guidance note	information in the standard added in order to increase the understanding of the requirements
hindcasting	a method using registered meteorological data to reproduce environmental parameters. Mostly used for reproducing wave parameters.
inspection	activities such as measuring, examination, testing, gauging one or more characteristics of an object or service and comparing the results with specified requirements to determine conformity
limit state	a state beyond which the structure no longer satisfies the requirements. The following categories of limit states are of relevance for structures: ULS = ultimate limit states FLS = fatigue limit states ALS = accidental limit states SLS = serviceability limit states.

Term	*Definition*
load and resistance factor design (LRFD)	method for design where uncertainties in loads are represented with a load factor and uncertainties in resistance are represented with a material factor
load effect	effect of a single design load or combination of loads on the equipment or system, such as stress, strain, deformation, displacement, motion, etc.
lowest mean daily average temperature	the lowest value on the annual mean daily average temperature curve for the area in question For temporary phases or restricted operations, the lowest mean daily average temperature may be defined for specific seasons. — Mean daily average temperature: the statistical mean average temperature for a specific calendar day. — Mean: statistical mean based on number of years of observations. — Average: average during one day and night.
lowest waterline	typical light ballast waterline for ships, wet transit waterline or inspection waterline for other types of units
non-destructive testing	structural tests and inspection of welds with radiography, ultrasonic or magnetic powder methods
object standard	the standards listed in Table 2
offshore installation	a general term for mobile and fixed structures, including facilities, which are intended for exploration, drilling, production, processing or storage of hydrocarbons or other related activities or fluids. The term includes installations intended for accommodation of personnel engaged in these activities. Offshore installation covers subsea installations and pipelines. The term does not cover traditional shuttle tankers, supply boats and other support vessels which are not directly engaged in the activities described above
operating conditions	conditions wherein a unit is on location for purposes of production, drilling or other similar operations, and combined environmental and operational loadings are within the appropriate design limits established for such operations (including normal operations, survival, accidental)
potential	the voltage between a submerged metal surface and a reference electrode
redundancy	the ability of a component or system to maintain or restore its function when a failure of a member or connection has occurred. Redundancy may be achieved for instance by strengthening or introducing alternative load paths
reference electrode	electrode with stable open-circuit potential used as reference for potential measurements
reliability	the ability of a component or a system to perform its required function without failure during a specified time interval
risk	the qualitative or quantitative likelihood of an accidental or unplanned event occurring considered in conjunction with the potential consequences of such a failure. In quantitative terms, risk is the quantified probability of a defined failure mode times its quantified consequence
service temperature	service temperature is a reference temperature on various structural parts of the unit used as a criterion for the selection of steel grades
serviceability limit states	corresponding to the criteria applicable to normal use or durability

Term	Definition
shakedown	a linear elastic structural behaviour is established after yielding of the material has occurred
slamming	impact load on an approximately horizontal member from a rising water surface as a wave passes. The direction of the impact load is mainly vertical
specified minimum yield strength	the minimum yield strength prescribed by the specification or standard under which the material is purchased
specified value	minimum or maximum value during the period considered. This value may take into account operational requirements, limitations and measures taken such that the required safety level is obtained.
splash zone	the external surfaces of the unit that are periodically in and out of the water. The determination of the splash zone includes evaluation of all relevant effects including influence of waves, tidal variations, settlements, subsidence and vertical motions, see Ch.2 Sec.9 [2.2].
submerged zone	the part of the unit which is below the splash zone, including buried parts
supporting structure	strengthening of the vessel structure, e.g. a deck, in order to accommodate loads and moments from a heavy or loaded object
survival condition	a condition during which a unit may be subjected to the most severe environmental loadings for which the unit is designed. Drilling or similar operations may have been discontinued due to the severity of the environmental loadings. The unit may be either afloat or supported on the sea bed, as applicable.
target safety level	a nominal acceptable probability of structural failure
temporary conditions	design conditions not covered by operating conditions, e.g. conditions during fabrication, mating and installation phases, transit phases, accidental
tensile strength	minimum stress level where strain hardening is at maximum or at rupture
transit conditions	all unit movements from one geographical location to another
unit	is a general term for an offshore installation such as ship shaped, column stabilised, self-elevating, tension leg or deep draught floater
utilisation factor	the fraction of anode material that can be utilised for design purposes
verification	examination to confirm that an activity, a product or a service is in accordance with specified requirements
ultimate limit states	corresponding to the maximum load carrying resistance

4 Abbreviations and symbols

4.1 Abbreviations

Abbreviations as shown in Table 6 are used in this standard.

Table 6 Abbreviations

Abbreviation	*Description*
AISC	American Institute of Steel Construction
ALS	accidental limit states
API	American Petroleum Institute
CN	classification note
CG	classification guideline
CTOD	crack tip opening displacement
DDF	deep draught floaters
DFF	design fatigue factor
EHS	extra high strength
FLS	fatigue limit state
FM	Fracture mechanics
HAT	highest astronomical tide
HISC	hydrogen induced stress cracking
HS	high strength
ISO	international organisation of standardisation
LAT	lowest astronomic tide
LMDAT	lowest mean daily average temperature
LRFD	load and resistance factor design
MPI	magnetic particle inspection
MSL	mean sea level
NACE	National Association of Corrosion Engineers
NDT	non-destructive testing
NS	normal strength
PWHT	post weld heat treatment
RP	recommended practise
RHS	rectangular hollow section
SCE	saturated calomel electrode
SCF	stress concentration factor
SLS	serviceability limit state
SMYS	specified minimum yield stress
SRB	sulphate reducing bacteria
SWL	Safe Working Load

Abbreviation	Description
TLP	tension leg platform
ULS	ultimate limit states
WSD	working stress design

4.2 Symbols

4.2.1 Latin characters

a_0	connection area
a_v	vertical acceleration
b	full breadth of plate flange
b_e	effective plate flange width
c	detail shape factor
d	bolt diameter
f	load distribution factor
f_E	elastic buckling stress
f_r	strength ratio
f_u	nominal lowest ultimate tensile strength
f_{ub}	ultimate tensile strength of bolt
f_w	strength ratio
f_y	specified minimum yield stress
g, g_o	acceleration due to gravity
h	height
h_{op}	vertical distance from the load point to the position of maximum filling height
k_a	correction factor for aspect ratio of plate field
k_m	bending moment factor
k_{pp}	fixation parameter for plate
k_{ps}	fixation parameter for stiffeners
k_s	hole clearance factor
k_t	shear force factor
l	stiffener span
l_o	distance between points of zero bending moments
n	number
p	pressure
p_d	design pressure

r	root face
r_c	radius of curvature
s	distance between stiffeners
t_0	net thickness of plate
t_k	corrosion addition
t_w	throat thickness
A_s	net area in the threaded part of the bolt
C	weld factor
C_e	factor for effective plate flange
D	deformation load
E	environmental load
F_d	design load
F_k	characteristic load
F_{pd}	design pre-loading force in bolt
G	permanent load
M	moment
M_p	plastic moment resistance
M_y	elastic moment resistance
N_p	number of supported stiffeners on the girder span
Ns	number of stiffeners between considered section and nearest support
P	load
P_{pd}	average design point load from stiffeners
Q	variable functional load
R	radius
R_d	design resistance
R_k	characteristic resistance
S	girder span as if simply supported
S_d	design load effect
S_k	characteristic load effect
SZ_l	lower limit of the splash zone
SZ_u	upper limit of the splash zone
W	steel with improved weldability
Z	steel grade with proven through thickness properties with respect to lamellar tearing.

4.2.2 Greek characters

α	angle between the stiffener web plane and the plane perpendicular to the plating
β_w	correlation factor
δ	deflection
φ	resistance factor
γ_f	load factor
γ_M	material factor (material coefficient)
γ_{Mw}	material factor for welds
λ	reduced slenderness
θ	rotation angle
μ	friction coefficient
ρ	density
σ_d	design stress
σ_{fw}	characteristic yield stress of weld deposit
σ_{jd}	equivalent design stress for global in-plane membrane stress
$\sigma_{pd\ 1}$	design bending stress
$\sigma_{pd\ 2}$	design bending stress
τ_d	design shear stress.

4.2.3 Subscripts

d	design value
k	characteristic value
p	plastic
y	yield.

CHAPTER 2 TECHNICAL CONTENT

SECTION 1 DESIGN PRINCIPLES

1 Introduction

1.1 General

1.1.1 This section describes design principles and design methods including:

— load and resistance factor design method
— design assisted by testing
— probability based design.

1.1.2 General design considerations regardless of design method are also given in [2.1] .

1.1.3 This standard is based on the load and resistance factor design method referred to as the LRFD method.

1.1.4 As an alternative or as a supplement to analytical methods, determination of load effects or resistance may in some cases be based either on testing or on observation of structural performance of models or full-scale structures.

1.1.5 Direct reliability analysis methods are mainly considered as applicable to special case design problems, to calibrate the load and resistance factors to be used in the LRFD method and for conditions where limited experience exists.

1.2 Aim of the design

Structures and structural elements shall be designed to:

— sustain loads liable to occur during all temporary, operating and damaged conditions, if required
— maintain acceptable safety for personnel and environment
— have adequate durability against deterioration during the design life of the structure.

2 General safety principles

2.1 General

2.1.1 This standard is built on the overall philosophy that structures shall provide a safety standard where structural failure is without substantial consequence.

2.1.2 Structures shall be designed to provide sufficient robustness to account for the consequence of:

— danger of loss of human life
— significant pollution
— major economic consequences.

2.1.3 The design of a structural system, its components and elements, shall account for the following principles:

- provide sufficient residual strength against total collapse in the case of structural failure of a vital element or component
- satisfactory resilience against relevant mechanical and chemical deterioration is achieved
- fabrication and construction complying with relevant standards, recognised techniques and practices
- in-service inspection, maintenance and associated principles for accessibility and repair are established.

2.1.4 Structural elements and components thereof shall possess ductile resistance, unless the specified purpose requires otherwise.

2.1.5 Structural connections shall in general be designed with the aim to minimise stress concentrations and reduce complex stress flow patterns.

2.1.6 Fatigue life improvements by methods such as grinding, hammer peening or TIG dressing of the weld shall not be used to provide increased fatigue life at the design stage. The fatigue life shall instead be extended by means of modification of the structural details. Where unavoidable, fatigue life improvement shall be limited to localized stress concentrations.

2.1.7 Transmission of high tensile stresses through the thickness of plates during welding, block assembly and operation shall be avoided as far as possible. In cases where transmission of high tensile stresses through thickness occur, structural material with proven through thickness properties shall be used. Object standards may give requirements where to use plates with proven through thickness properties.

2.1.8 Structures that are not complying with [2.1.3] to [2.1.7] shall be subject to special consideration and acceptance.

3 Limit states

3.1 General

3.1.1 A limit state is a condition beyond which a structure or a part of a structure exceeds a specified design requirement.

3.1.2 The following limit states are considered in this standard:

Table 1 Limit states

Limit states	*Definition*
Ultimate limit states (ULS)	Corresponding to the ultimate resistance for carrying loads
Fatigue limit states (FLS)	Related to the possibility of failure due to the effect of cyclic loading
Accidental limit states (ALS)	Corresponding to damage to components due to an accidental event or operational failure
Serviceability limit states (SLS)	Corresponding to the criteria applicable to normal use or durability

3.1.3 Examples of limit states within each category:

3.1.3.1 Ultimate limit states (ULS)

- loss of structural resistance (excessive yielding and buckling)

— failure of components due to brittle fracture
— loss of static equilibrium of the structure, or of a part of the structure, considered as a rigid body, e.g. overturning or capsizing
— failure of critical components of the structure caused by exceeding the ultimate resistance (in some cases reduced by repeated loads) or the ultimate deformation of the components
— transformation of the structure into a mechanism (collapse or excessive deformation).

3.1.3.2 Fatigue limit states (FLS)

— cumulative damage due to repeated loads.

3.1.3.3 Accidental limit states (ALS)

— structural damage caused by accidental loads
— ultimate resistance of damaged structures
— maintain structural integrity after local damage or flooding
— loss of station keeping (free drifting).

3.1.3.4 Serviceability limit states (SLS)

— deflections that may alter the effect of the acting forces
— deformations that may change the distribution of loads between supported rigid objects and the supporting structure
— excessive vibrations producing discomfort or affecting non-structural components
— motion that exceed the limitation of equipment
— temperature induced deformations.

4 Design by LRFD method

4.1 General

4.1.1 Design by the LRFD method is a design method by which the target safety level is obtained as closely as possible by applying load and resistance factors to characteristic reference values of the basic variables. The basic variables are, in this context, defined as:

— loads acting on the structure
— resistance of the structure or resistance of materials in the structure.

4.1.2 The target safety level is achieved by using deterministic factors representing the variation in load and resistance and the reduced probabilities that various loads will act simultaneously at their characteristic values.

4.2 The load and resistance factor design format (LRFD)

4.2.1 The level of safety of a structural element is considered to be satisfactory if the design load effect (S_d) does not exceed the design resistance (R_d):

$$S_d \leq R_d$$

The equation: $S_d = R_d$, defines a limit state.

4.2.2 A design load is obtained by multiplying the characteristic load by a given load factor:

$$F_d = \gamma_f F_k$$

where:

F_d = design load

γ_f = load factor

F_k = characteristic load, see Sec.2.

The load factors and combinations for ULS, ALS, FLS and SLS shall be applied according to [4.3] to [4.7].

4.2.3 A design load effect is the most unfavourable combined load effect derived from the design loads, and may, if expressed by one single quantity, be expressed by:

$$S_d = q(F_{d1},\dots,F_{dn})$$

where:

S_d = design load effect

q = load effect function.

4.2.4 If the relationship between the load and the load effect is linear, the design load effect may be determined by multiplying the characteristic load effects by the corresponding load factors:

$$S_d = \sum_{i=1}^{n} (\gamma_{fi} S_{ki})$$

where:

S_{ki} = characteristic load effect.

4.2.5 In this standard the values of the resulting material factor are given in the respective sections for the different limit states.

4.2.6 The resistance for a particular load effect is, in general, a function of parameters such as structural geometry, material properties, environment and load effects (interaction effects).

4.2.7 The design resistance (R_d) is determined as follows:

$$R_d = \phi R_k$$

where:

R_k = characteristic resistance

φ = resistance factor.

The resistance factor relate to the material factor γ_M as follows:

$$\phi = \frac{1}{\gamma_M}$$

where:

γ_M = material factor.

4.2.8 R_k may be calculated on the basis of characteristic values of the relevant parameters or determined by testing. Characteristic values should be based on the 5th percentile of the test results.

4.2.9 Load factors account for:

— possible unfavourable deviations of the loads from the characteristic values
— the reduced probability that various loads acting together will act simultaneously at their characteristic value
— uncertainties in the model and analysis used for determination of load effects.

4.2.10 Material factors account for:

— possible unfavourable deviations in the resistance of materials from the characteristic values
— possible reduced resistance of the materials in the structure, as a whole, as compared with the characteristic values deduced from test specimens.

4.3 Characteristic load

4.3.1 The representative values for the different groups of limit states in the operating design conditions shall be based on Sec.2:

— For the ULS load combination, the representative value corresponding to a load effect with an annual probability of exceedance equal to, or less than, 10^{-2} (100 years).
— For the ALS load combination for damaged structure, the representative load effect is determined as the most probable annual maximum value.
— For the FLS, the representative value is defined as the expected load history.
— For the SLS, the representative value is a specified value, dependent on operational requirements.

4.3.2 For the temporary design conditions, the characteristic values may be based on specified values, which shall be selected dependent on the measurers taken to achieve the required safety level. The value may be specified with due attention to the actual location, season of the year, weather forecast and consequences of failure.

4.4 Load factors for ULS

4.4.1 For analysis of ULS, two sets of load combinations shall be used when combining design loads as defined in Table 2. The combinations denoted a) and b) shall be considered in both operating and temporary conditions. The load factors are generally applicable for all types of structures, but other values may be specified in the respective object standards.

Table 2 Load factors γ_f for ULS

Combination of design loads	*Load categories*			
	G	*Q*	*E*	*D*
a)	1.3	1.3	0.7	1.0
b)	1.0	1.0	1.3	1.0

Load categories are:
G = permanent load
Q = variable functional load
E = environmental load
D = deformation load

For description of load categories see Sec.2.

4.4.2 When permanent loads (G) and variable functional loads (Q) are well defined, e.g. hydrostatic pressure, a load factor of 1.2 may be used in combination a) for these load categories.

4.4.3 If a load factor γ_f = 1.0 on G and Q loads in combination a) results in higher design load effect, the load factor of 1.0 shall be used.

4.4.4 Based on a safety assessment considering the risk for both human life and the environment, the load factor γ for environmental loads may be reduced to 1.15 in combination b) if the structure is unmanned during extreme environmental conditions.

4.5 Load factor for FLS

4.5.1 The structure shall be able to resist expected fatigue loads, which may occur during temporary and operation design conditions. Where significant cyclic loads may occur in other phases, e.g. wind excitation during fabrication, such cyclic loads shall be included in the fatigue load estimates.

4.5.2 The load factor γ_f in the FLS is 1.0 for all load categories.

4.6 Load factor for SLS

For analyses of SLS the load factor γ_f is 1.0 for all load categories, both for temporary and operating design conditions.

4.7 Load factor for ALS

The load factors γ_f in the ALS is 1.0.

5 Design assisted by testing

5.1 General

5.1.1 Design by testing or observation of performance shall in general be supported by analytical design methods.

5.1.2 Load effects, structural resistance and resistance against material degradation may be established by means of testing or observation of the actual performance of full-scale structures.

5.2 Full-scale testing and observation of performance of existing structures

Full-scale tests or monitoring on existing structures may be used to give information on response and load effects to be utilised in calibration and updating of the safety level of the structure.

6 Probability based design

6.1 Definition

The structural reliability, or structural safety, is defined as the probability that failure will not occur or that a specified criterion will not be exceeded within a specified period of time.

6.2 General

6.2.1 As an alternative to design using the LRFD method specified in this standard, a full probability-based design using a structural reliability analysis may be carried out. This requires a recognized structural reliability method to be used.

6.2.2 This subsection gives requirements for structural reliability analysis undertaken in order to document compliance with the offshore standards.

6.2.3 Acceptable procedures for reliability analyses are documented in the DNV-CN-30.6.

6.2.4 Reliability analyses shall be based on level 3 reliability methods. These methods utilise the probability of failure as a measure and require knowledge of the distribution of all basic variables.

6.2.5 In this standard, level 3 reliability methods are mainly considered applicable to:

— calibration of level 1 method to account for improved knowledge. (Level 1 methods are deterministic analysis methods that use only one characteristic value to describe each uncertain variable, i.e. the LRFD method applied in the standards)
— special case design problems
— novel designs where limited or no experience exists.

6.2.6 Reliability analysis may be updated by utilising new information. Where such updating indicates that the assumptions upon which the original analysis was based are not valid, and the result of such non-validation is deemed to be essential to safety, the subject's approval may be revoked.

6.2.7 Target reliabilities shall be commensurate with the consequence of failure. The method of establishing such target reliabilities, and the values of the target reliabilities themselves, should be agreed in each separate case. To the extent possible, the minimum target reliabilities shall be based on established cases that are known to have adequate safety.

6.2.8 Where well-established cases do not exist, e.g. in the case of novel and unique design solution, the minimum target reliability values shall be based upon one or a combination of the following considerations:

— transferable target reliabilities similar existing design solutions
— internationally recognised codes and standards
— DNV-CN-30.6.

6.2.9 Suitably competent and qualified personnel shall carry out the structural reliability analysis. Any extension into new areas of application shall be subject to technical verification.

SECTION 2 LOADS AND LOAD EFFECTS

1 Introduction

1.1 General

The requirements in this section define and specify load components and load combinations to be considered in the overall strength analysis as well as design pressures applicable in formulae for local design.

1.2 Scope

1.2.1 Impact pressure caused by the sea (slamming, bow impact) or by liquid cargoes in partly filled tanks (sloshing) are not covered by this section.

1.2.2 For structural arrangement of mooring equipment and arrangement/devices for towing, see DNVGL-OS-E301 Ch.2 Sec.4 [15] and DNVGL-OS-E301 Ch.2 Sec.4 [16]. The mooring and towing equipment, including the support to main structure, shall be designed for the loads and acceptance criteria specified in DNVGL-OS-E301 Ch.2 Sec.4.

2 Basis for selection of characteristic loads

2.1 General

2.1.1 Unless specific exceptions apply, as documented within this standard, the characteristic loads documented in Table 1 and Table 2 shall apply in the temporary and operating design conditions, respectively.

2.1.2 Where environmental and accidental loads may act simultaneously, the characteristic loads may be determined based on their joint probability distribution.

Table 1 Basis for selection of characteristic loads for temporary design conditions

Load category	*Limit states – temporary design conditions*				
	ULS	*FLS*	*ALS*		*SLS*
			Intact structure	*Damaged structure*	
Permanent (G)	Expected value				
Variable (Q)	Specified value				
Environmental (E)	Specified value	Expected load history	Specified value	Specified value	Specified value
Accidental (A)			Specified value		
Deformation (D)	Expected extreme value				
For definitions, see Ch.1 Sec.1. See DNVGL-ST-N001 and VMO standard DNV-OS-H206)					

Table 2 Basis for selection of characteristic loads for operating design conditions

Load category	*Limit states – operating design conditions*				
	ULS	*FLS*	*ALS*		*SLS*
			Intact structure	*Damaged structure*	
Permanent (G)	Expected value				
Variable (Q)	Specified value				
Environmental (E)	Annual probability[1)] being exceeded $= 10^{-2}$ (100 year return period)	Expected load history	Not applicable	Load with return period not less than 1 year	Specified value
Accidental (A)			Specified value, see also Ch.2 Sec.6		
Deformation (D)	Expected extreme value				
1) The joint probability of exceedance applies, see [6].					

3 Permanent loads (G)

3.1 General

3.1.1 Permanent loads are loads that will not vary in magnitude, position or direction during the period considered. Examples are:

— mass of structure
— mass of permanent ballast and equipment
— external and internal hydrostatic pressure of a permanent nature
— reaction to the above e.g. articulated tower base reaction.

3.1.2 The characteristic load of a permanent load is defined as the expected value based on accurate data of the unit, mass of the material and the volume in question.

4 Variable functional loads (Q)

4.1 General

4.1.1 Variable functional loads are loads which may vary in magnitude, position and direction during the period under consideration, and which are related to operations and normal use of the installation.

4.1.2 Examples are:

— personnel
— stored materials, equipment, gas, fluids and fluid pressure
— crane operational loads
— loads from fendering

— loads associated with installation operations
— loads associated with drilling operations
— loads from variable ballast and equipment
— variable cargo inventory for storage vessels
— helicopters.

4.1.3 The characteristic value of a variable functional load is the maximum (or minimum) specified value, which produces the most unfavourable load effects in the structure under consideration.

4.1.4 The specified value shall be determined on the basis of relevant specifications. An expected load history shall be used in FLS.

4.2 Variable functional loads on deck areas

Variable functional loads on deck areas of the topside structure and modules shall be based on Table 3 unless specified otherwise in the design basis or the design brief. The intensity of the distributed loads depends on local or global aspects as shown in Table 3. The following notations are used:

Notation	*Example*
Local design	Design of plates, stiffeners, and brackets
Primary design	Design of girders (beams) and beam-columns
Global design	Design of topside main load bearing structure and substructure

Table 3 Variable functional loads on deck areas

	Local design		*Primary design*	*Global design 1)*
	Distributed load, p (kN/m²)	*Point load, P (kN)*	*Apply factor to distributed load, p*	*Apply factor to distributed load, p*
Storage areas	*q*	1.5 *q*	1.0	1.0
Lay down areas	*q*	1.5 *q*	*f*	*f*
Lifeboat platforms	9.0	9.0	1.0	may be ignored
Area between equipment	5.0	5.0	*f*	may be ignored
Walkways, staircases and platforms, crew spaces	4.5	4.5	*f*	may be ignored
Walkways and staircases for inspection only	3.0	3.0	*f*	may be ignored
Areas not exposed to other functional loads	2.5	2.5	1.0	-

Notes:

- Wheel loads shall be added to the distributed loads where relevant (wheel loads may be assumed acting on an area of 300 mm x 300 mm).
- Point loads to be applied on an area 100 mm x 100 mm, and at the most severe position, but not added to wheel loads or distributed loads.
- q to be evaluated for each case. Storage areas should not be designed for less than 13 kN/m^2. Lay down areas should not be designed for less than 15 kN/m^2.
- $f = \min\{1.0 ; (0.5 + 3/ \sqrt{A})\}$, where A is the loaded area in m^2.
- 1) Global load cases should be established based upon worst case, characteristic load combinations, complying with the limiting global criteria to the structure. For buoyant structures these criteria are established by requirements for the floating position in still water, and intact and damage stability requirements, as documented in the operational manual, considering variable load on the deck and in tanks.

  Variable functional loads shall be considered in global structural analysis. For capacity checks of the structural elements, the global stresses and the local stresses from tank pressure loads, weigh of equipments, etc, should be combined.

Guidance note:

If the table is used in connection with design of e.g. accommodation structure or topside modules with several decks the following applies:

- Each local deck area (plates and stiffeners) shall be designed using loads for local design.
- For strength control of primary design structure like girders, columns and support, the variable functional load may be reduced with a factor f as a function of the area A, as maximum functional load on each individual deck will not appear simultaneously.

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

4.3 Tank pressures

4.3.1 The structure shall be designed to resist the maximum hydrostatic pressure of the heaviest filling in tanks that may occur during fabrication, installation and operation.

4.3.2 Hydrostatic pressures in tanks should be based on a minimum density equal to that of seawater, ρ = 1.025 t/m^3. Tanks for higher density fluids, e.g. mud, shall be designed on basis of special consideration. The density, upon which the scantlings of individual tanks are based, shall be given in the operating manual.

4.3.3 Pressure loads that may occur during emptying of water or oil filled structural parts for condition monitoring, maintenance or repair shall be evaluated.

4.3.4 Hydrostatic pressure heads shall be based on tank filling arrangement by e.g. pumping, gravitational effect, accelerations as well as venting arrangements.

4.3.5 Pumping pressures may be limited by installing appropriate alarms and auto-pump cut-off system (i.e. high level and high-high level with automatic stop of the pumps). In such a situation the pressure head may be taken to be the cut-off pressure head. Descriptions and requirements related to different tank arrangements are given in DNVGL-OS-D101 Ch.2 Sec.3 [3.3] for ballast tanks, and in DNVGL-OS-D101 Ch.2 Sec.3 [5.2] for other tanks.

4.3.6 Dynamic pressure heads due to flow through pipes shall be considered, see [4.3.8].

4.3.7 All tanks shall be designed for the following internal design pressure:

$$p_d = \rho \cdot g_0 \cdot h_{op} \cdot \left(\gamma_{f,G,Q} + \frac{a_v}{g_0} \cdot \gamma_{f,E}\right) \quad (kN/m^2)$$

where:

a_v = maximum vertical acceleration (m/s^2), being the coupled motion response applicable to the tank in question

h_{op} = vertical distance (m) from the load point to the position of maximum filling height.
For tanks adjacent to the sea that are situated below the extreme operational draught, the maximum filling height should not be taken lower than the extreme operational draught.

ρ = density of liquid (t/m^3)

g_0 = 9.81 m/s^2

$\gamma_{f,G,Q}$ = load factor for ULS, permanent and functional loads

$\gamma_{f,E}$ = load factor for ULS, environmental loads.

4.3.8 For tanks where the air pipe may be filled during filling operations, the following additional internal design pressure conditions shall be considered:

$$p_d = (\rho \cdot g_0 \cdot h_{op} + p_{dyn}) \cdot \gamma_{f,G,Q} \ (\text{kN/m}^2)$$

where:

h_{op} = vertical distance (m) from the load point to the position of maximum filling height.
For tanks adjacent to the sea that are situated below the extreme operational draught, the maximum filling height should not be taken lower than the extreme operational draught

p_{dyn} = pressure (kN/m^2) due to flow through pipes, minimum 25 kN/m^2

$\gamma_{f,G,Q}$ = load factor for ULS, permanent and functional loads.

4.3.9 In a situation where design pressure head might be exceeded, this should be considered an ALS condition.

5 Environmental loads (E)

5.1 General

5.1.1 Environmental loads are loads which may vary in magnitude, position and direction during the period under consideration, and which are related to operations and normal use of the installation. Examples are:

— hydrodynamic loads induced by waves and current
— inertia forces
— wind
— earthquake
— tidal effects
— marine growth
— snow and ice.

5.1.2 Practical information regarding environmental loads and conditions are given in DNVGL-RP-C205.

5.2 Environmental loads for mobile offshore units

5.2.1 The design of mobile offshore units shall be based on the most severe environmental loads that the structure may experience during its design life. The applied environmental conditions shall be defined in the design basis or design brief, and stated in the unit's operation manual.

5.2.2 The North Atlantic scatter diagram should be used in ULS, ALS and FLS for unrestricted world wide operation.

5.3 Environmental loads for site specific units

5.3.1 The parameters describing the environmental conditions shall be based on observations from or in the vicinity of the relevant location and on general knowledge about the environmental conditions in the area. Data for the joint occurrence of e.g. wave, wind and current conditions should be applied.

5.3.2 According to this standard, the environmental loads shall be determined with stipulated probabilities of exceedance. The statistical analysis of measured data or simulated data should make use of different statistical methods to evaluate the sensitivity of the result. The validation of distributions with respect to data should be tested by means of recognised methods.

5.3.3 The analysis of the data shall be based on the longest possible time period for the relevant area. In the case of short time series the statistical uncertainty shall be accounted for when determining design values. Hindcasting may be used to extend measured time series, or to interpolate to places where measured data have not been collected. If hindcasting is used, the model shall be calibrated against measured data, to ensure that the hindcast results comply with available measured data.

5.4 Determination of characteristic hydrodynamic loads

5.4.1 Hydrodynamic loads shall be determined by analysis. When theoretical predictions are subjected to significant uncertainties, theoretical calculations shall be supported by model tests or full scale measurements of existing structures or by a combination of such tests and full scale measurements.

5.4.2 Hydrodynamic model tests should be carried out to:

— confirm that no important hydrodynamic feature has been overlooked by varying the wave parameters (for new types of installations, environmental conditions, adjacent structure, etc.)
— support theoretical calculations when available analytical methods are susceptible to large uncertainties
— verify theoretical methods on a general basis.

5.4.3 Models shall be sufficient to represent the actual installation. The test set-up and registration system shall provide a basis for reliable, repeatable interpretation.

5.4.4 Full-scale measurements may be used to update the response prediction of the relevant structure and to validate the response analysis for future analysis. Such tests may especially be applied to reduce uncertainties associated with loads and load effects which are difficult to simulate in model scale.

5.4.5 In full-scale measurements it is important to ensure sufficient instrumentation and logging of environmental conditions and responses to ensure reliable interpretation.

5.4.6 Wind tunnel tests should be carried out when:

— wind loads are significant for overall stability, offset, motions or structural response
— there is a danger of dynamic instability.

5.4.7 Wind tunnel test may support or replace theoretical calculations when available theoretical methods are susceptible to large uncertainties, e.g. due to new type of installations or adjacent installation influence the relevant installation.

5.4.8 Theoretical models for calculation of loads from icebergs or drift ice should be checked against model tests or full-scale measurements.

5.4.9 Proof tests of the structure may be necessary to confirm assumptions made in the design.

5.4.10 Hydrodynamic loads on appurtenances (anodes, fenders, strakes etc,) shall be taken into account, when relevant.

5.5 Wave loads

5.5.1 Wave theory or kinematics shall be selected according to recognised methods with due consideration of actual water depth and description of wave kinematics at the surface and the water column below.

5.5.2 Linearised wave theories, e.g. airy, may be used when appropriate. In such circumstances the influence of finite amplitude waves shall be taken into consideration.

5.5.3 Wave loads should be determined according to DNVGL-RP-C205.

5.5.4 For large volume structures where the wave kinematics is disturbed by the presence of the structure, typical radiation or diffraction analyses shall be performed to determine the wave loads, e.g. excitation forces or pressures.

5.5.5 For slender structures (typically chords and bracings, tendons, risers) where the Morison equation is applicable, the wave loads should be estimated by selection of drag and inertia coefficients as specified in DNVGL-RP-C205.

5.5.6 In the case of adjacent large volume structures disturbing the free field wave kinematics, the presence of the adjacent structures may be considered by radiation and diffraction analyses for calculation of the wave kinematics.

5.6 Wave induced inertia forces

5.6.1 The load effect from inertia forces shall be taken into account in the design. Examples where inertia forces can be of significance are:

— heavy objects
— tank pressures
— flare towers
— drilling towers
— crane pedestals.

5.6.2 The accelerations shall be based on direct calculations or model tests unless specified in the object standards.

5.7 Wind loads

5.7.1 The wind velocity at the location of the installation shall be established on the basis of previous measurements at the actual and adjacent locations, hindcast predictions as well as theoretical models and other meteorological information. If the wind velocity is of significant importance to the design and existing wind data are scarce and uncertain, wind velocity measurements should be carried out at the location in question.

5.7.2 Characteristic values of the wind velocity should be determined with due account of the inherent uncertainties.

Guidance note:

Wind loads may be determined in accordance with DNVGL-RP-C205.

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

5.7.3 The pressure acting on vertical external bulkheads exposed to wind shall not be taken less than 2.5 kN/m^2 unless otherwise documented.

5.7.4 For structures being sensitive to dynamic loads, for instance tall structures having long natural period of vibration, the stresses due to the gust wind pressure considered as static shall be multiplied by an appropriate dynamic amplification factor.

5.8 Vortex induced oscillations

Consideration of loads from vortex shedding on individual elements due to wind, current and waves may be based on DNVGL-RP-C205. Vortex induced vibrations of frames shall also be considered. The material and structural damping of individual elements in welded steel structures shall not be set higher than 0.15% of critical damping.

5.9 Current

Characteristic current design velocities shall be based upon appropriate consideration of velocity or height profiles and directionality.

Guidance note:

Further details regarding current design loads are given in DNVGL-RP-C205.

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

5.10 Tidal effects

5.10.1 For floating structures constrained by tendon mooring systems, tidal effects can significantly influence the structure's buoyancy and the mean loads in the mooring components. Therefore the choice of tide conditions for static equilibrium analysis is important. Tidal effects shall be considered in evaluating the various responses of interest. Higher mean water levels tend to increase maximum mooring tensions, hydrostatic loads, and current loads on the hull, while tending to decrease under deck wave clearances.

5.10.2 These effects of tide may be taken into account by performing a static balance at the various appropriate tide levels to provide a starting point for further analysis, or by making allowances for the appropriate tide level in calculating extreme responses.

5.11 Marine growth

5.11.1 Marine growth is a common designation for a surface coating on marine structures, caused by plants, animals and bacteria. In addition to the direct increase in structure weight, marine growth may cause an increase in hydrodynamic drag and added mass due to the effective increase in member dimensions, and may alter the roughness characteristics of the surface.

5.11.2 Effect of marine growth shall be considered, where relevant.

5.12 Snow and ice accumulation

5.12.1 Ice accretion from sea spray, snow, rain and air humidity shall be considered, where relevant.

5.12.2 Snow and ice loads may be reduced or neglected if a snow and ice removal procedures are established.

5.12.3 When determining wind and hydrodynamic load, possible increases of cross-sectional area and changes in surface roughness caused by icing shall be considered, where relevant.

5.12.4 For buoyant structures the possibility of uneven distribution of snow and ice accretion shall be considered.

5.13 Direct ice load

5.13.1 Where impact with sea ice or icebergs may occur, the contact loads shall be determined according to relevant, recognised theoretical models, model tests or full-scale measurements.

5.13.2 When determining the magnitude and direction of the loads, the following factors shall be considered:

— geometry and nature of the ice
— mechanical properties of the ice
— velocity and direction of the ice
— geometry and size of the ice and structure contact area
— ice failure mode as a function of the structure geometry

— environmental forces available to drive the ice
— inertia effects for both ice and structure.

5.14 Water level, settlements and erosion

5.14.1 When determining water level in the calculation of loads, the tidal water and storm surge shall be taken into account. Calculation methods that take into account the effects that the structure and adjacent structures have on the water level shall be used.

5.14.2 Uncertainty of measurements and possible erosion shall be considered.

5.15 Earthquake

5.15.1 Relevant earthquake effects shall be considered for bottom fixed structures.

5.15.2 Earthquake excitation design loads and load histories may be described either in terms of response spectra or in terms of time histories. For the response spectrum method all modes of vibration which contribute significantly to the response shall be included. Correlation effects shall be accounted for when combining the modal response maximum.

5.15.3 When performing time-history earthquake analysis, the response of the structure and foundation system shall be analysed for a representative set of time histories. Such time histories shall be selected and scaled to provide a best fit of the earthquake motion in the frequency range where the main dynamic response is expected.

5.15.4 The dynamic characteristics of the structure and its foundation should be determined using a three-dimensional analytical model. A two-dimensional or axis-symmetric model may be used for the soil and structure interaction analysis provided compatibility with the three-dimensional structural model is ensured.

5.15.5 Where characteristic ground motions, soil characteristics, damping and other modelling parameters are subject to great uncertainties, a parameter sensitivity study should be carried out.

5.15.6 Consideration shall be given to the possibility that earthquakes in the local region may cause other effects such as subsea earthslides, critical pore pressure built-up in the soil or major soil deformations affecting foundation slabs, piles or skirts.

6 Combination of environmental loads

6.1 General

6.1.1 Where applicable data are available joint probability of environmental load components, at the specified probability level, may be considered. Alternatively, joint probability of environmental loads may be approximated by combination of characteristic values for different load types as shown in Table 4.

6.1.2 Generally, the long-term variability of multiple loads may be described by a scatter diagram or joint density function including information about direction. Contour curves may then be derived which give combination of environmental parameters, which approximately describe the various loads corresponding to the given probability of exceedance.

6.1.3 Alternatively, the probability of exceedance may be referred to the load effects. This is particularly relevant when direction of the load is an important parameter.

6.1.4 For bottom founded and symmetrical moored structures it is normally conservative to consider co-linear environmental loads. For certain structures, such as moored ship shaped units, where the colinear assumption is not conservative, non colinear criteria should be used.

6.1.5 The load intensities for various types of loads may be selected to correspond to the probabilities of exceedance as given in Table 4.

6.1.6 In a short-term period with a combination of waves and fluctuating wind, the individual variations of the two load processes should be assumed uncorrelated.

Table 4 Proposed combinations of different environmental loads in order to obtain ULS combinations with 10^{-2} annual probability of exceedance and ALS loads with return period not less than 1 year

Limit state	*Wind*	*Waves*	*Current*	*Ice*	*Sea level*
ULS	10^{-2}	10^{-2}	10^{-1}		10^{-2}
	10^{-1}	10^{-1}	10^{-2}		10^{-2}
	10^{-1}	10^{-1}	10^{-1}	10^{-2}	Mean water level
ALS	Return period not less than 1 year	Return period not less than 1 year	Return period not less than 1 year		Return period not less than 1 year

7 Accidental loads (A)

7.1 General

7.1.1 Accidental loads are loads related to abnormal operations or technical failure. Relevant accidental events are given in Sec.6.

7.1.2 Relevant accidental loads should be determined on the basis of an assessment and relevant experience. Guidance regarding implementation, use and updating of such assessments and generic accidental loads, see DNVGL-OS-A101.

7.1.3 For temporary design conditions, the characteristic load may be a specified value dependent on practical requirements. The level of safety related to the temporary design conditions shall not be inferior to the safety level required for the operating design conditions.

8 Deformation loads (D)

8.1 General

Deformation loads are loads caused by inflicted deformations such as:

— temperature loads
— built-in deformations
— settlement of foundations

— tether pre-tension on a TLP.

8.2 Temperature loads

8.2.1 Structures shall be designed for the most extreme temperature differences they may be exposed to. This applies to, but is not limited to:

— storage tanks
— structural parts that are exposed to radiation from the top of a flare boom. For flare boom radiation a one hour mean wind with a return period of 1 year may be used to calculate the spatial flame extent and the air cooling in the assessment of heat radiation from the flare boom
— structural parts that are in contact with pipelines, risers or process equipment.

8.2.2 The ambient sea or air temperature is calculated as an extreme value with an annual probability of exceedance equal to 10^{-2} (100 years).

8.3 Settlements and subsidence of sea bed

8.3.1 Settlement of the foundations into the sea bed shall be considered for permanently located bottom founded units.

8.3.2 The possibility of, and the consequences of, subsidence of the seabed as a result of changes in the subsoil and in the production reservoir during the design life of the unit, shall be considered.

8.3.3 Reservoir settlements and subsequent subsidence of the seabed shall be calculated as a conservatively estimated mean value.

9 Load effect analysis

9.1 General

9.1.1 Load effects, in terms of motions, displacements, or internal forces and stresses of the structure, shall be determined with due regard for:

— spatial and temporal nature, including:
  — possible non-linearities of the load
  — dynamic character of the response.
— relevant limit states for design check
— desired accuracy in the relevant design phase.

9.1.2 Permanent-, functional-, deformation-, and fire-loads should be treated by static methods of analysis. Environmental (wave, wind and earthquake) loads and certain accidental loads (impacts, explosions) may require dynamic analysis. Inertia and damping forces are important when the periods of steady-state loads are close to natural periods or when transient loads occur.

9.1.3 In general, three frequency bands shall be considered for offshore structures:

High frequency (HF)	Rigid body natural periods below dominating wave periods (typically ringing and springing responses in TLP's).
Wave frequency (WF)	Area with wave periods in the range 4 to 25 s typically. Applicable to all offshore structures located in the wave active zone.
Low frequency (LF)	This frequency band relates to slowly varying responses with natural periods above dominating wave energy (typically slowly varying surge and sway motions for column-stabilised and ship-shaped units as well as slowly varying roll and pitch motions for deep draught floaters).

9.1.4 A global wave motion analysis is required for structures with at least one free mode. For fully restrained structures a static or dynamic wave-structure-foundation analysis is required.

9.1.5 Uncertainties in the analysis model are expected to be taken care of by the load and resistance factors. If uncertainties are particularly high, conservative assumptions shall be made.

9.1.6 If analytical models are particularly uncertain, the sensitivity of the models and the parameters utilised in the models shall be examined. If geometric deviations or imperfections have a significant effect on load effects, conservative geometric parameters shall be used in the calculation.

9.1.7 In the final design stage theoretical methods for prediction of important responses of any novel system should be verified by appropriate model tests. See Sec.1 [5.2].

9.1.8 Earthquake loads need only be considered for restrained modes of behaviour. See object standards for requirements related to the different objects.

9.2 Global motion analysis

The purpose of a motion analysis is to determine displacements, accelerations, velocities and hydrodynamic pressures relevant for the loading on the hull and superstructure, as well as relative motions (in free modes) needed to assess airgap and green water requirements. Excitation by waves, current and wind should be considered.

9.3 Load effects in structures and soil or foundation

9.3.1 Displacements, forces or stresses in the structure and foundation, shall be determined for relevant combinations of loads by means of recognised methods, which take adequate account of the variation of loads in time and space, the motions of the structure and the limit state which shall be verified. Characteristic values of the load effects shall be determined.

9.3.2 Non-linear and dynamic effects associated with loads and structural response shall be accounted for whenever relevant.

9.3.3 The stochastic nature of environmental loads shall be adequately accounted for.

9.3.4 Description of the different types of analyses are covered in the various object standards.

SECTION 3 STRUCTURAL CATEGORISATION, MATERIAL SELECTION AND INSPECTION PRINCIPLES

1 Scope

This section describes the structural categorisation, selection of steel materials and inspection principles to be applied in design and construction of offshore steel structures.

2 Temperatures for selection of material

2.1 General

2.1.1 The design temperature for a unit is the reference temperature for assessing areas where the unit can be transported, installed and operated. The design temperature shall be lower or equal to the lowest mean daily average temperature in air (LMDAT) for the relevant areas. For seasonal restricted operations the LMDAT for the season may be applied.

2.1.2 The service temperatures for different parts of a unit apply for selection of structural steel.

2.1.3 The service temperature for various structural parts is given in [2.2] and [2.3]. In case different service temperatures are defined in [2.2] and [2.3] for a structural part the lower specified value shall be applied. Further details regarding service temperature for different structural elements are given in the various object standards.

2.1.4 In all cases where the temperature is reduced by localised cryogenic storage or other cooling conditions, such factors shall be taken into account in establishing the service temperatures for considered structural parts.

2.2 Floating units

2.2.1 External structures above the lowest waterline shall be designed with service temperature not higher than the design temperature for the area(s) where the unit shall operate.

2.2.2 External structures below the lowest waterline need not be designed for service temperatures lower than 0°C. A higher service temperature may be accepted if adequate supporting data can be presented relative to lowest mean daily average temperature applicable to the relevant actual water depths.

2.2.3 Internal structures in way of permanently heated rooms need not be designed for service temperatures lower than 0°C.

2.3 Bottom fixed units

2.3.1 External structures above the lowest astronomical tide (LAT) shall be designed with service temperature not higher than the design temperature.

2.3.2 Materials in structures below the lowest astronomical tide (LAT) need not be designed for service temperatures lower than 0°C.

A higher service temperature may be accepted if adequate supporting data can be presented relative to lowest mean daily average temperature applicable to the relevant actual water depths.

3 Structural category

3.1 General

The purpose of the structural categorisation is to ensure adequate material qualities and suitable inspection to avoid brittle fracture. The purpose of inspection is also to remove defects that may grow into fatigue cracks during the units design life.

Guidance note:

Conditions that may result in brittle fracture are sought avoided. Brittle fracture may occur under a combination of:

— presence of sharp defects such as cracks
— high tensile stress in direction normal to planar defect(s)
— material with low fracture toughness.

High stresses in a component may occur due to welding. A complex connection is likely to provide more restraint and larger residual stress than a simple one. This residual stress may be partly removed by post weld heat treatment if necessary. Also a complex connection shows a more three-dimensional stress state due to external loading than simple connections. This stress state may provide basis for a cleavage fracture.

The fracture toughness is dependent on temperature and material thickness. These parameters are accounted for separately in selection of material. The resulting fracture toughness in the weld and the heat affected zone is also dependent on the fabrication method.

Thus, to avoid brittle fracture, first a material with suitable fracture toughness for the actual service temperature and thickness is selected. Then a proper fabrication method is used. In special cases post weld heat treatment may be performed to reduce crack driving stresses, see [4.5] and DNVGL-OS-C401. A suitable amount of inspection is carried out to remove planar defects larger than that are acceptable. In this standard selection of material with appropriate fracture toughness and avoidance of unacceptable defects are achieved by linking different types of connections to different structural categories and inspection categories.

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

3.2 Selection of structural category

3.2.1 Components are classified into structural categories according to the following criteria:

— significance of component in terms of consequence of failure
— stress condition at the considered detail that together with possible weld defects or fatigue cracks may provoke brittle fracture.

Guidance note:

The consequence of failure may be quantified in terms of residual strength of the structure when considering failure of the actual component.

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

3.2.2 Structural category for selection of materials shall be determined according to principles given in Table 1.

Table 1 Structural categories for selection of materials 1)

Structural category	*Principles for determination of structural category*
Special	Structural parts where failure will have substantial consequences and are subject to a stress condition that may increase the probability of a brittle fracture 2).
Primary	Structural parts where failure will have substantial consequences.
Secondary	Structural parts where failure will be without significant consequence.

Structural category	Principles for determination of structural category
1) Examples of determination of structural categories are given in the various object standards. 2) In complex joints a triaxial or biaxial stress pattern will be present. This may give conditions for brittle fracture where tensile stresses are present in addition to presence of defects and material with low fracture toughness.	

3.3 Inspection of welds

3.3.1 Requirements for type and extent of inspection are given in DNVGL-OS-C401 dependent on assigned inspection category for the welds. The requirements are based on the consideration of fatigue damage and assessment of general fabrication quality.

3.3.2 The inspection category is by default related to the structural category according to Table 2.

Table 2 Inspection categories

Inspection category	Structural category
I	Special
II	Primary
III	Secondary

3.3.3 The weld connection between two components shall be assigned an inspection category according to the highest of the joined components. For stiffened plates, the weld connection between stiffener and stringer and girder web to the plate may be inspected according to inspection category III.

3.3.4 If the fabrication quality is assessed by testing, or well known quality from previous experience, the extent of inspection required for elements within structural category primary may be reduced, but not less than for inspection category III.

3.3.5 Fatigue critical details within structural category primary and secondary shall be fabricated according to tolerances for structural category special, and inspected according to requirements in category I.

3.3.6 Critical welds that are not accessible for inspection and repair during operation shall be inspected according to requirements in category I.

3.3.7 The extent of NDT for welds in block joints and erection joints transverse to main stress direction shall not be less than for inspection category II.

4 Structural steel

4.1 General

4.1.1 Where the subsequent requirements for steel grades are dependent on plate thickness, these are based on the nominal thickness as built.

4.1.2 The requirements in this subsection deal with the selection of various structural steel grades in compliance with the requirements given in DNVGL-OS-B101. Where other, agreed codes or standards have been utilised in the specification of steels, the application of such steel grades within the structure shall be specially considered.

4.1.3 The steel grades selected for structural components shall be related to calculated stresses and requirements to toughness properties. Requirements for toughness properties are in general based on the Charpy V-notch test and are dependent on service temperature, structural category and thickness of the component in question.

4.1.4 The material toughness may also be evaluated by fracture mechanics testing in special cases, see [4.4] and DNVGL-OS-C401.

4.1.5 In structural cross-joints where high tensile stresses are acting perpendicular to the plane of the plate, the plate material shall be tested to prove the ability to resist lamellar tearing, Z-quality, see [4.2.3].

4.1.6 Requirements for forging and castings are given in DNVGL-OS-B101.

4.2 Material designations

4.2.1 Structural steel of various strength groups are referred to in Table 3.

4.2.2 Each strength group consists of two parallel series of steel grades:

— steels of normal weldability
— steels of improved weldability, that is, steels with leaner chemistry and better weldability.

Extra high strength steels (EHS) consists of an additional series of steel grades:

— offshore grade steels, that is, steels where the indicated minimum yield stress is independent of the product thickness.

For more details, see DNVGL-OS-B101.

Table 3 Material designations

Designation	*Strength group*	*Specified minimum yield stress* f_y *(N/mm^2)*
VL	Normal strength steel (NS)	235
VL W		190 - 235[1]
VL 27S	High strength steel (HS)	265
VL W27S		220 - 265[1]
VL 32		315
VL W32		270 - 315[1]
VL 36		355
VL W36		310 - 355[1]
VL 40		390
VL 420 and VL W420	Extra high strength steel (EHS)[2]	365 - 420[1]
VL O420		420
VL 460 and VL W460		390 - 460[1]

Designation	Strength group	Specified minimum yield stress f_y (N/mm^2)
VL O460		460
VL 500 and VL W500		440 - 500 1)
VL O500		500
VL 550		490 - 550 1)
VL O550		550
VL 620		560 - 620 1)
VL O620		620
VL 690		630 - 690 1)
VL O690		690
VL 890 3)		
VL 960 3)		

1) For steels of improved weldability the required specified minimum yield stress is reduced for increasing material thickness, see DNVGL-OS-B101.
2) Three series of extra strength steels are specified, see DNVGL-OS-B101 Ch.2 Sec.2 [4.1].
3) The steel is generally not permitted in hull structure. Shall not be used without advance approval by the purchaser and responsible verifying authority (DNV GL in case of classification/certification).

4.2.3 Different steel grades are defined within each strength group, depending upon the required impact toughness properties. The grades are referred to as A, B, D, E, and F for normal weldability, and AW, BW, DW, and EW for improved weldability and offshore grades AO, DO, EO and FO with yield stress independent of production thickness, as shown in Table 4.

Additional symbol:

Z = steel grade of proven through-thickness properties. This symbol is omitted for steels of improved weldability although improved through-thickness properties are required.

Table 4 Applicable steel grades

Strength group	Grade			Test temperature 3) (ºC)
	Normal weldability	Improved weldability 2)	Offshore grades	
NS	A	-		Not tested
	B 1)	BW		0
	D	DW		-20
	E	EW		-40
HS	A	AW		0
	D	DW		-20
	E	EW		-40
	F	-		-60

Strength group	*Grade*			*Test temperature* [3] *(ºC)*
	Normal weldability	*Improved weldability* [2]	*Offshore grades*	
EHS	A	-	AO	0
	D	DW	DO	-20
	E	EW	EO	-40
	F	-	FO	-60

1) Charpy V-notch tests are required for thickness above 25 mm but are subject to agreement between the contracting parties for thickness of 25 mm or less.
2) For steels with improved weldability, through thickness properties are specified, see DNVGL-OS-B101.
3) Charpy V-notch impact tests, see DNVGL-OS-B101.

4.3 Selection of structural steel

The grade of steel to be used shall in general be related to the service temperature and thickness for the applicable structural category as shown in Table 5.

Table 5 Thickness limitations (mm) of structural steels for different structural categories and service temperatures (ºC)

Structural Category	*Strength group*	*Grade*	*≥ 10*	*0*	*-10*	*-20*	*-25*	*-30*
Secondary	NS	A	35	30	25	20	15	10
		B/BW	70	60	50	40	30	20
		D/DW	150	150	100	80	70	60
		E/EW	150	150	150	150	120	100
	HS	A/AW	60	50	40	30	20	15
		D/DW	120	100	80	60	50	40
		E/EW	150	150	150	150	120	100
		F	150	150	150	150	*)	*)
	EHS	A/AO	70	60	50	40	30	20
		D/DW/DO	150	150	100	80	70	60
		E/EW/EO	150	150	150	150	120	100
		F/FO	150	150	150	150	*)	*)
Primary	NS	A	30	20	10	N.A.	N.A.	N.A.
		B/BW	40	30	25	20	15	10
		D/DW	70	60	50	40	35	30
		E/EW	150	150	100	80	70	60
	HS	A/AW	30	25	20	15	12.5	10
		D/DW	60	50	40	30	25	20
		E/EW	120	100	80	60	50	40
		F	150	150	150	150	*)	*)

Structural Category	*Strength group*	*Grade*	*≥ 10*	*0*	*-10*	*-20*	*-25*	*-30*
	EHS	A/AO	35	30	25	20	17.5	15
		D/DW/DO	70	60	50	40	35	30
		E/EW/EO	150	150	100	80	70	60
		F/FO	150	150	150	150	*)	*)
Special	NS	D/DW	35	30	25	20	17.5	15
		E/EW	70	60	50	40	35	30
	HS	A/AW	15	10	N.A.	N.A.	N.A.	N.A.
		D/DW	30	25	20	15	12.5	10
		E/EW	60	50	40	30	25	20
		F	120	100	80	60	50	40
	EHS	A/AO	20	15	10	N.A.	N.A.	N.A.
		D/DW/DO	35	30	25	20	17.5	15
		E/EW/EO	70	60	50	40	35	30
		F/FO	150	150	100	80	70	60

*) For service temperature below -20°C, the limit to be specially considered.

N.A. = no application

In order to obtain thickness limitation for intermediate service temperature, linear interpolation may be used.

4.3.1 Selection of a better steel grade than minimum required in design shall not lead to more stringent requirements in fabrication.

4.3.2 Grade of steel to be used for thickness less than 10 mm and/or service temperature above 10 °C may be specially considered.

4.3.3 Welded steel plates and sections of thickness exceeding the upper limits for the actual steel grade as given in Table 5 shall be evaluated in each individual case with respect to the fitness for purpose of the weldments. The evaluation should be based on fracture mechanics testing and analysis, see [4.4].

4.3.4 For structural parts subjected to compressive and/or low tensile stresses, consideration may be given to the use of lower steel grades than stated in Table 5.

4.3.5 The use of steels with specified minimum yield stress greater than 550 N/mm^2 (VL 550) shall be subject to special consideration for applications where anaerobic environmental conditions such as stagnant water, organically active mud (bacteria) and hydrogen sulphide may predominate.

4.3.6 Predominantly anaerobic conditions can for this purpose be characterised by a concentration of sulphate reducing bacteria (SRB) in the order of magnitude $>10^3$ SRB/ml (method according to NACE TPC publication no.3).

4.3.7 The steels' susceptibility to hydrogen induced stress cracking (HISC) shall be specially considered when used for critical applications (such as jack-up legs and spud cans). See also Sec.9.

4.3.8 Doublers, sleeves and foundations welded to structure which belongs to primary and special category shall in general be assigned to the same structural category and strength group as the structural member it is welded to.

Substitutive material may be accepted upon special consideration. It is then required that the doubler plate is delivered with 3.1 material certificate. The weld between the doubler and the base plate shall follow the inspection requirements given in [3]. Substitutive material is not allowed for doublers welded to:

— special category structure
— plates forming boundaries to sea.

4.4 Fracture mechanics testing

For units which are intended to operate continuously at the same location for more than 5 years, fracture mechanics (FM) testing shall be included in the qualification of welding procedures for joints which all of the following apply:

— the design temperature is lower than +10ºC
— the joint is in special area
— at least one of the adjoining members is fabricated from steel with a SMYS larger than or equal to 420 MPa.

For details on FM testing methods, see DNVGL-OS-C401 Ch.2 Sec.5 [3.3.7].

4.5 Post weld heat treatment

For units which are intended to operate continuously at the same location for more than 5 years, post weld heat treatment (PWHT) shall be applied for joints in C-Mn steels in special areas when the material thickness at the welds exceeds 50 mm. For details, see DNVGL-OS-C401 Ch.2 Sec.6 [9.6]. However, if satisfactory performance in the as-welded condition can be documented by a fitness-for-purpose assessment applying fracture mechanics testing, fracture mechanics and fatigue crack growth analyses, PWHT may be omitted.

SECTION 4 ULTIMATE LIMIT STATES

1 General

1.1 General

1.1.1 This section gives provisions for checking of ultimate limit states for typical structural elements used in offshore steel structures.

1.1.2 The ultimate strength capacity (yield and buckling) of structural elements shall be assessed using a rational, justifiable, engineering approach.

1.1.3 Structural capacity checks of all structural components shall be performed. The capacity checks shall consider both excessive yielding and buckling.

1.1.4 Simplified assumptions regarding stress distributions may be used provided the assumptions are made in accordance with generally accepted practice, or in accordance with sufficiently comprehensive experience or tests.

1.1.5 Gross scantlings may be utilised in the calculation of hull structural strength, provided a corrosion protection system in accordance with Sec.9 is installed and maintained.

1.1.6 In case corrosion protection in accordance with Sec.9 is not installed (and maintained) corrosion additions as given in Sec.9 [2.4.7] shall be used. The corrosion addition shall not be accounted for in the determination of stresses and resistance for local capacity checks.

1.2 Structural analysis

1.2.1 The structural analysis may be carried out as linear elastic, simplified rigid-plastic, or elastic-plastic analyses. Both first order or second order analyses may be applied. In all cases, the structural detailing with respect to strength and ductility requirement shall conform to the assumption made for the analysis.

1.2.2 When plastic or elastic-plastic analyses are used for structures exposed to cyclic loading, e.g. wave loads, checks shall be carried out to verify that the structure will shake down without excessive plastic deformations or fracture due to repeated yielding. A characteristic or design cyclic load history shall be defined in such a way that the structural reliability in case of cyclic loading, e.g. storm loading, is not less than the structural reliability for ULS for non-cyclic loads.

1.2.3 In case of linear analysis combined with the resistance formulations set down in this standard, shakedown can be assumed without further checks.

1.2.4 If plastic or elastic-plastic structural analyses are used for determining the sectional stress resultants, limitations to the width thickness ratios apply. Relevant width thickness ratios are found in the relevant codes used for capacity checks.

1.2.5 When plastic analysis and/or plastic capacity checks are used (cross section type I and type II, according to App.A), the members shall be capable of forming plastic hinges with sufficient rotation capacity to enable the required redistribution of bending moments to develop. It shall also be checked that the load pattern will not be changed due to the deformations.

1.2.6 Cross sections of beams are divided into different types dependent of their ability to develop plastic hinges. A method for determination of cross sectional types is found in App.A.

1.3 Ductility

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

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

— Unstable fracture caused by a combination of the following factors: brittle material, low temperature in the steel, a design resulting in high local stresses and the possibilities for weld defects.
— Structural details where ultimate resistance is reached with plastic deformations only in limited areas, making the global behaviour brittle.
— Shell buckling.
— Buckling where interaction between local and global buckling modes occurs.

1.4 Yield check

1.4.1 Structural members for which excessive yielding are possible modes of failure shall be investigated for yielding. Individual design stress components and the von Mises equivalent design stress for plated structures shall not exceed the design resistance (Sec.1 [4.2]).

Guidance note:

a) For plated structures the von Mises equivalent design stress is defined as follows:

$$\sigma_{jd} = \sqrt{\sigma_{xd}^2 + \sigma_{yd}^2 - \sigma_{xd}\sigma_{yd} + 3\tau_d^2}$$

where σ_{xd} and σ_{yd} are design membrane stresses in x- and y-direction respectively, τ_d is design shear stress in the x-y plane (i.e. local bending stresses in plate thickness not included).

b) For von Mises check of plated panels the design resistance factor for loading condition a) and b) as given in Sec.1 [4.2.7] may be increased with a factor of 1.1, provided each individual nominal stress component (σ_{xd}, σ_{yd}, τ_d) satisfies the design resistance without such increase. See IACS regulation concerning mobile offshore drilling units D3.4.3 and D3.5.

c) In case local plate bending stresses are of importance for yield check, e.g for lateral loaded plates, yield check may be performed according to DNV-RP-C201 Pt.1 Sec.5.

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

1.4.2 Local peak stresses from linear elastic analysis in areas with pronounced geometrical changes, may exceed the yield stress provided the adjacent structural parts has capacity for the redistributed stresses.

Guidance note:

a) Areas above yield determined by a linear finite element method analysis may give an indication of the actual area of plastification. Otherwise, a non-linear finite element method analysis may be carried out in order to trace the full extent of the plastic zone.

b) The yield checks do not refer to local stress concentrations in the structure or to local modelling deficiencies in the finite element model.

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

1.4.3 For yield check of welded connections, see Sec.8.

1.5 Buckling check

1.5.1 Elements of cross sections not fulfilling requirements to cross section type III shall be checked for local buckling. Cross sectional types are defined in App.A.

1.5.2 Buckling analysis shall be based on the characteristic buckling resistance for the most unfavourable buckling mode.

1.5.3 It shall be ensured that there is conformity between the initial imperfections in the buckling resistance formulas and the tolerances given in the applied fabrication standard.

Guidance note:

If buckling resistance is calculated in accordance with DNVGL-RP-C201 for plated structures, DNVGL-RP-C202 for shells, or DNVGL-CG-0128 Sec.3 [5] for bars and framework, the tolerance requirements given in DNVGL-OS-C401 should not be exceeded, unless specifically documented.

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

2 Flat plated structures and stiffened panels

2.1 General

The material factor γ_M for plated structures is 1.15.

2.2 Yield check

2.2.1 Yield check of plating and stiffeners is given in [6].

2.2.2 Yield check of girders is given in [7].

2.3 Buckling check

Buckling check of plated structures is given in DNVGL-RP-C201.

2.4 Capacity checks according to other than DNV GL codes

2.4.1 Stiffeners and girders may be designed according to provisions for beams in recognised standards such as Eurocode 3.

2.4.2 Material factors when using Eurocode 3 are given in Table 1.

Table 1 Material factors used with Eurocode 3

Type of calculation	*Material factor* [1)]	*Value*
Resistance of class 1, 2 or 3 cross sections	γ_{M0}	1.15
Resistance of class 4 cross sections	γ_{M1}	1.15
Resistance of members to buckling	$\gamma_{M\ 1}$	1.15
1) Symbols according to Eurocode 3.		

2.4.3 Plates, stiffeners and girders may alternatively be checked according to NORSOK N-004.

3 Shell structures

3.1 General

3.1.1 The buckling stability of cylindrical and un-stiffened conical shell structures may be checked according to DNVGL-RP-C202.

3.1.2 For interaction between shell buckling and column buckling, DNVGL-RP-C202 may be used.

3.1.3 If DNVGL-RP-C202 is applied, the material factor for shells shall be in accordance with Table 2.

Table 2 Material factors γ_M for buckling

Type of structure	*$\lambda \leq 0.5$*	*$0.5 < \lambda < 1.0$*	*$\lambda \geq 1.0$*
Shells of single curvature (cylindrical shells, conical shells, rings and/or stiffeners)	1.15	$0.85 + 0.60\ \lambda$	1.45
Note that the slenderness is based on the buckling mode under consideration. λ = reduced slenderness parameter $\sqrt{\frac{f_y}{f_E}}$ f_y = specified minimum yield stress f_E = elastic buckling stress for the buckling mode under consideration.			

4 Tubular members, tubular joints and conical transitions

4.1 General

4.1.1 Tubular members may be checked according to DNVGL-CG-0128 Sec.3 [5], ISO 19902, API RP 2A or NORSOK N-004. For interaction between local shell buckling and column buckling, and effect of external pressure, DNVGL-RP-C202 may be used.

4.1.2 Cross sections of tubular member are divided into different types dependent of their ability to develop plastic hinges and resist local buckling. Effect of local buckling of slender cross sections shall be considered.

Guidance note:

a) Effect of local buckling of tubular members without external pressure (i.e. subject to axial force and/or bending moment) are given in App.A cross section type IV. DNVGL-RP-C202 [3.8] may be used, see [3.1].

b) Effect of local buckling of tubular members with external pressure need not be considered for the following diameter (D) to thickness (t) ratio:

$$\frac{D}{t} \leq 0.5\sqrt{\frac{E}{f_y}}$$

where:

E = modulus of elasticity, and

f_y = specified minimum yield stress.

In case of local shell buckling, see [3.1], DNVGL-RP-C202 [3.8]. API RP 2A - LRFD or NORSOK N-004 may be used.

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

4.1.3 Tubular joints and conical transitions may be checked according to ISO 19902 API RP 2A or NORSOK N-004.

4.1.4 The material factor γ_M for tubular structures is 1.15.

5 Non-tubular beams, columns and frames

5.1 General

5.1.1 The design of members shall take into account the possible limits on the resistance of the cross section due to local buckling.

Guidance note:

Cross sections of member are divided into different types dependent of their ability to develop plastic hinges and resist local buckling, see App.A. In case of local buckling, i.e. for cross sectional type IV, DNV-RP-C201 may be used.

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

5.1.2 Buckling checks may be performed according to DNVGL-CG-0128 Sec.3 [5].

5.1.3 Capacity check may be performed according to recognised standards such as Eurocode 3 or AISC Manual of Steel Construction (9th edition).

Guidance note:

In order to keep the same format when use of AISC, non-tubular profiles shall be checked according to AISC 9th edition (1989) as recommended in API RP 2A (22nd edition).

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

5.1.4 The material factors according to Table 1 shall be used if Eurocode 3 is used for calculation of structural resistance.

6 Special provisions for plating and stiffeners

6.1 Scope

The requirements given below are minimum scantlings to plate and stiffened panels with respect to yield. Dimensions and further references with respect to buckling capacity are given in [2].

6.2 Minimum thickness

The thickness of plates should not be less than:

$$t = \frac{14.3 t_0}{\sqrt{f_{yd}}} \quad (mm)$$

where:

f_{yd} = design yield strength f_y/γ_M

f_y is the minimum yield stress (N/mm^2) as given in Sec.3 Table 3

t_0 = 7 mm for primary structural elements

= 5 mm for secondary structural elements

γ_M = material factor for steel

= 1.15.

6.3 Bending of plating

The thickness of plating subjected to lateral pressure shall not be less than:

$$t = \frac{15{,}8 k_a s \sqrt{p_d}}{\sqrt{\sigma_{pd1} k_{pp}}} \quad (mm)$$

where:

k_a = correction factor for aspect ratio of plate field

= $(1.1 - 0.25\ s/l)^2$

= maximum 1.0 for s/l = 0.4

= minimum 0.72 for s/l = 1.0

s = stiffener spacing (m), measured along the plating

p_d = design pressure (kN/m^2) as given in Sec.2

σ_{pd1} = design bending stress (N/mm^2), taken as the smaller of:

- 1.3 ($f_{yd} - \sigma_{jd}$), and
- $f_{yd} = f_y / \gamma_M$

σ_{jd} = equivalent design stress for in-plane membrane stress:

$$\sigma_{jd} = \sqrt{\sigma_{xd}^2 + \sigma_{yd}^2 - \sigma_{xd}\sigma_{yd} + 3\tau_d^2}$$

k_{pp} = fixation parameter for plate

= 1.0 for clamped edges

= 0.5 for simply supported edges.

Guidance note:

The design bending stress σ_{pd1} is given as a bi-linear capacity curve for the plate representing the remaining capacity of plate when reduced for in-plane membrane stress.

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

6.4 Stiffeners

6.4.1 The section modulus Z_s for longitudinals, beams, frames and other stiffeners subjected to lateral pressure shall not be less than:

$$Z_s = \frac{l^2 s p_d}{k_m \sigma_{pd2} k_{ps}} 10^6 (mm^3), \ minimum\ 15 \cdot 10^3\ (mm^3)$$

where:

l = stiffener span (m)

k_m = bending moment factor, see Table 3

σ_{pd2} = design bending stress (N/mm^2)

= $f_{yd} - \sigma_{jd}$

k_{ps} = fixation parameter for stiffeners

= 1.0 if at least one end is clamped

= 0.9 if both ends are simply supported.

Guidance note:

The section modulus Z_s will typically be calculated for plate side and stiffener flange side. The design bending stress σ_{pd2} for plate side is given as a linear capacity curve for the plate representing the remaining capacity of plate when reduced for in-plane membrane stress (longitudinal, transverse and shear stresses). The design bending stress for stiffener flange side is given as a linear capacity curve for the stiffener flange representing the remaining capacity of stiffener when reduced for longitudinal membrane stress.

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

6.4.2 The requirement in [6.4.1] applies to an axis parallel to the plating. For stiffeners at an oblique angle with the plating an approximate requirement to standard section modulus may be obtained by multiplying the section modulus from [6.4.1] with the factor:

$$\frac{1}{\cos \alpha}$$

where:

α = angle between the stiffener web plane and the plane perpendicular to the plating.

6.4.3 Stiffeners with sniped ends may be accepted where dynamic stresses are small and vibrations are considered to be of small importance, provided that the plate thickness supported by the stiffener is not less than:

$$t \geq 16 \sqrt{\frac{(l - 0.5s) s p_d}{f_{yd}}} \quad (mm)$$

In such cases the section modulus of the stiffener calculated as indicated in [6.4.1] shall normally be based on the following parameter values:

k_m = 8

k_{ps} = 0.9

The stiffeners should normally be snipped with an angle of maximum 30º.

Guidance note:

For typical sniped end detail as described above, a stress range lower than 30 MPa can be considered as a small dynamic stress.

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

6.4.4 The shear area A_s for longitudinals, beams, frames and other stiffeners subjected to lateral pressure shall not be less than:

$$A_s = \frac{lsp_d}{2\tau_{pds}} 10^3 (mm^2)$$

where the design shear stress (in N/mm^2) is given by:

$$\tau_{pds} = 0,577\sqrt{(f_{yd})^2 - (\sigma_{xd})^2}$$

Guidance note:

The design shear stress, τ_{pds}, is given as a quadratic capacity curve for the stiffener representing the remaining capacity when reduced for in-plane membrane axial stress. For typical stiffeners the shear area, A_s, is given by the web of the stiffener. For stiffeners subject to high shear loads the combined effect of stiffener bending and shear loads should be considered by e.g a corresponding reduction in the web when checking Z_s according to [6.4.1].

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

7 Special provisions for girder and girder systems

7.1 Scope

7.1.1 The requirements in this section give minimum scantlings to simple girders with respect to yield. Further, procedures for the calculations of complex girder systems are indicated.

7.1.2 Dimensions and further references with respect to buckling capacity are given in [2].

7.2 Minimum thickness

The thickness of web and flange plating shall not be less than given in [6.2] and [6.3].

7.3 Bending and shear

7.3.1 The requirements for section modulus and web area are applicable to simple girders supporting stiffeners or other girders exposed to linearly distributed lateral pressure. It is assumed that the girder satisfies the basic assumptions of simple beam theory, and that the supported members are approximately evenly spaced and has similar support conditions at both ends. Other loads shall be specially considered.

7.3.2 When boundary conditions for individual girders are not predictable due to dependence of adjacent structures, direct calculations according to the procedures given in [7.7] shall be carried out.

7.3.3 The section modulus and web area of the girder shall be taken in accordance with particulars as given in [7.4] and [7.5]. Structural modelling in connection with direct stress analysis shall be based on the same particulars when applicable.

7.4 Effective flange

The effective plate flange area is defined as the cross sectional area of plating within the effective flange width. The cross section area of continuous stiffeners within the effective flange may be included. The effective flange width b_e is determined by the following formula:

$$b_e = C_e b$$

where:

C_e	=	parameter given in Figure 1 for various numbers of evenly spaced point loads (N_p) on the girder span
b	=	full breadth of plate flange (m), e.g. span of the supported stiffeners, or distance between girders, see also [7.6.2].
l	=	S for simply supported girders
	=	0.6 S for girders fixed at both ends
S	=	girder span as if simply supported, see also [7.6.2].

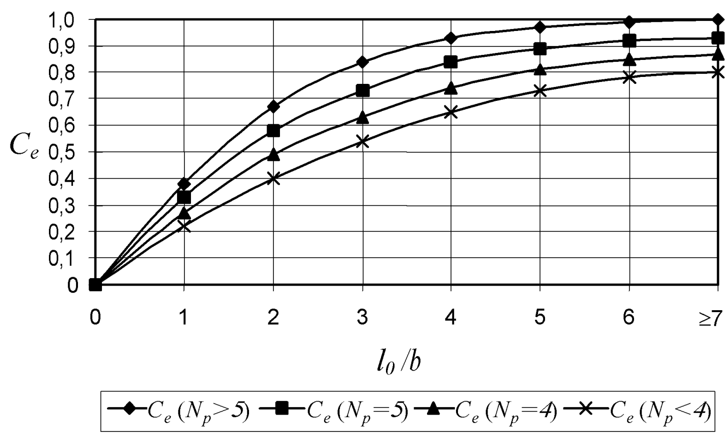


Figure 1 Graphs for the effective flange parameter C_e

7.5 Effective web

Holes in girders will generally be accepted provided the shear stress level is acceptable and the buckling capacity and fatigue life is documented to be sufficient.

7.6 Strength requirements for simple girders

7.6.1 Simple girders subjected to lateral pressure and which are not taking part in the overall strength of the structure, shall comply with the following minimum requirements:

— section modulus according to [7.6.2]
— web area according to [7.6.3].

7.6.2 Section modulus Z_g

$$Z_g = \frac{S^2 b p_d}{k_m \sigma_{pd2}} 10^6 \ (mm^3)$$

S	=	girder span (m). The web height of in-plane girders may be deducted. When brackets are fitted at the ends, the girder span S may be reduced by two thirds of the bracket arm length, provided the girder ends may be assumed clamped and provided the section modulus at the bracketed ends is satisfactory
b	=	breadth of load area (m) (plate flange), b may be determined as:
	=	0.5 (l_1 + l_2), l_1 and l_2 are the spans of the supported stiffeners on both sides of the girder, respectively, or distance between girders
p_d	=	design pressure (kN/m^2) as given in Sec.2
k_m	=	bending moment factor, see [7.6.4]
σ_{pd2}	=	design bending stress (N/mm^2), see [6.4.1]
σ_{jd}	=	equivalent design stress for global in-plane membrane stress.

$$\sigma_{jd} = \sqrt{\sigma_{xd}^2 + \sigma_{yd}^2 - \sigma_{xd}\sigma_{yd} + 3\tau_d^2}$$

7.6.3 Web area A_w

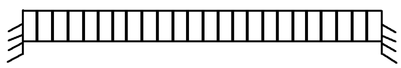
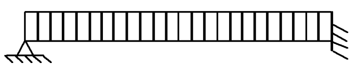
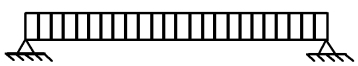
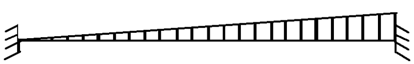
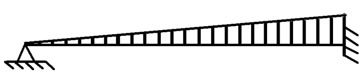
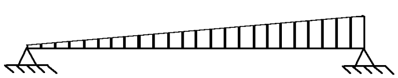
$$A_w = \frac{k_\tau S b p_d - N_s P_{pd}}{\tau_{pd}} 10^3 \ (mm^2)$$

where:

k_τ	=	shear force factor, see [7.6.4].
N_s	=	number of stiffeners between considered section and nearest support the N_s-value shall in no case be taken greater than (N_p+1)/4.
N_p	=	number of supported stiffeners on the girder span
P_{pd}	=	average design point load (kN) from stiffeners between considered section and nearest support
τ_{pd}	=	0.5 f_{yd} (N/mm^2).

7.6.4 The k_m- and k_τ-values referred to in [7.6.2] and [7.6.3] may be calculated according to general beam theory. In Table 3 k_m- and k_τ-values are given for some defined load and boundary conditions. Note that the smallest k_m-value shall be applied to simple girders. For girders where brackets are fitted or the flange area has been partly increased due to large bending moment, a larger k_m-value may be used outside the strengthened region.

Table 3 Values of k_m and k_T

Load and boundary conditions	*Bending moment and shear force factors*		
Positions: *1* Support, *2* Field, *3* Support	*1* k_{m1} k_{T1}	*2* k_{m2} -	*3* k_{m3} k_{T3}
	12 0.5	24 -	12 0.5
	- 0.38	14.2 -	8 0.63
	- 0.5	8 -	- 0.5
	15 0.3	23.3 -	10 0.7
	- 0.2	16.8 -	7.5 0.8
	- 0.33	7.8 -	- 0.67

7.7 Complex girder system

7.7.1 For girders that are parts of a complex 2 or 3 dimensional structural system, a complete structural analysis shall be carried out.

7.7.2 Calculation methods or computer programs applied shall take into account the effects of bending, shear, axial and torsional deformation.

7.7.3 The calculations shall reflect the structural response of the 2 or 3 dimensional structure considered, with due attention to boundary conditions.

7.7.4 For systems consisting of slender girders, calculations based on beam theory (frame work analysis) may be applied, with due attention to:

- shear area variation, e.g. cut-outs
- moment of inertia variation
- effective flange
- lateral buckling of girder flanges.

7.7.5 The most unfavourable of the loading conditions given in Sec.2 shall be applied.

7.7.6 For girders taking part in the overall strength of the unit, stresses due to the design pressures given in Sec.2 shall be combined with relevant overall stresses.

SECTION 5 FATIGUE LIMIT STATES

1 General

1.1 General

1.1.1 In this standard, requirements are given in relation to fatigue analyses based on fatigue tests and fracture mechanics. See DNVGL-RP-C203 and DNVGL-CG-0129 for practical details with respect to fatigue design of offshore structures. See also Sec.1 [2.1.6].

1.1.2 The aim of fatigue design is to ensure that the structure has an adequate fatigue life. Calculated fatigue lives should also form the basis for establishing efficient inspection programmes during fabrication and the operational life of the structure.

1.1.3 The resistance against fatigue is normally given as S-N curves, i.e. stress range (S) versus number of cycles to failure (N) based on fatigue tests. Fatigue failure should be defined as when the crack has grown through the thickness.

1.1.4 The S-N curves shall in general be based on a 97.6% probability of survival, corresponding to the mean-minus-two-standard-deviation curves of relevant experimental data.

1.1.5 The design fatigue life for the structure shall account for design fatigue factors (DFF) in order to reduce the probability of fatigue failure during the units design life.

1.1.6 To ensure that the structure will fulfil the intended function, a fatigue assessment shall be carried out for each individual element or component which is subjected to fatigue loading. Where appropriate, the fatigue assessment shall be supported by a detailed fatigue analysis. It shall be noted that any element or component of the structure, hereof any welded joint and attachment or other form of stress concentration is potentially a source of fatigue cracking and should be individually considered.

1.1.7 The analyses shall be performed utilising relevant site specific environmental data for the area(s) in which the unit will be operated.

1.1.8 For world wide operation the North Atlantic scatter diagram shall be used as defined in DNV-RP-C205.

1.2 Design fatigue factors

1.2.1 Design fatigue factors (DFF) shall be applied to the design life to reduce the probability for fatigue failure. The calculated fatigue life shall be longer than the design life times the DFF.

1.2.2 The DFF are dependent on the significance of the structural components with respect to structural integrity and availability for inspection and repair.

1.2.3 The design fatigue factors in Table 1 shall be used for structures complying with the basis given in Ch.2 Sec.1 [2.1.3] to Ch.2 Sec.1 [2.1.7]. Structures not complying with these basis shall be specially considered.

DNV GL AS

Table 1 Design fatigue factors (DFF)

	DFF related to survey cycle	
Structural element	5-year inspection interval, carried out in dry dock	5-year inspection interval, carried out afloat
External structure, accessible for regular inspection and repair in dry and clean conditions	1	1
External structure, accessible for inspection but not accessible for repair in dry and clean conditions	1	2 [1) 2)]
Internal structure, accessible and not welded directly to the submerged shell plate	1	1
Internal structure, accessible and welded directly to the submerged shell plate	1	2
Non-accessible areas, not planned to be accessible for inspection and repairs during operation	3	3

[1)] For units that are planned to be inspected afloat at a sheltered location:

- DFF of 1 from 1 m above lowest inspection waterline and upwards.
- DFF of 2 from 1 m above the lowest inspection waterline and downwards.

[2)] For units intended for prolonged stay at location:

- DFF of 1 above the splash zone.
- DFF of 2 below the splash zone.
- DFF of 3 in the splash zone.

Splash zone is the area that not is accessible, due to typically waves and current. The splash zone shall be defined for the unit, as relevant.

Guidance note:

1) Where the likely crack propagation develops from a location which is accessible for inspection and repair to a structural element having no access, such location should itself be deemed to have the same categorisation as the most demanding category when considering the most likely crack path. For example, a weld detail on the inside (dry space) of a submerged shell plate should be allocated the same DFF as that relevant for a similar weld located externally on the plate.

2) The DFF for external structure includes buttwelds/seams in the plate.

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

1.2.4 Structures below 150 m from water level are subject to special consideration.

1.2.5 The various object standards define the design fatigue factors to be applied for typical structural details. In case of conflict, the object standards take precedence over this standard.

1.3 Methods for fatigue analysis

1.3.1 The fatigue analysis should be based on S-N data, determined by fatigue testing of the considered welded detail, and the linear damage hypothesis. When appropriate, the fatigue analysis may alternatively be based on fracture mechanics.

1.3.2 In fatigue critical areas where the fatigue life is estimated based on simplified methods is below the acceptable limit, more accurate investigation or a fracture mechanics analysis shall be performed.

1.3.3 For calculations based on fracture mechanics, it shall be documented that the in-service inspections accommodate a sufficient time interval between time of crack detection and the time of unstable fracture. See DNVGL-RP-C203 for more details.

1.3.4 All significant stress ranges, which contribute to fatigue damage in the structure, shall be considered. The long term distribution of stress ranges may be found by deterministic or spectral analysis. Dynamic effects shall be duly accounted for when establishing the stress history.

SECTION 6 ACCIDENTAL LIMIT STATES

1 General

1.1 General

1.1.1 Accidental limit states (ALS) shall be assessed for all relevant accidental events. Satisfactory protection against accidental damage shall in principle be obtained by:

— low damage probability
— acceptable damage consequences.

1.1.2 The ALS shall be checked in the following steps:

1) The structure withstand the design accidental loads caused by the design accidental events, as found applicable for the unit in intact condition.
2) In case of failure in step 1), the unit shall be able to resist a 1 year environmental condition, without loss of floatability, stability or global structural integrity.

1.1.3 Design accidental loads and safety principles are given in DNVGL-OS-A101.

Relevant accidental events specific for the unit types are given in the respective object standard, as applicable.

1.1.4 For applicable accidental loads, both the load factor (γ_f) and a material factor (γ_M) shall be 1.0. For shell structures the material factors (γ_M) given in Sec.4 [3] Table 2, may be divided by a factor of 1.15.

1.1.5 Non-linear static and dynamic FE analysis may be applied for the strength calculation. All relevant failure modes (e.g. strain rate, local buckling, joint overloading) shall be checked. Local overloading of the structural capacity is acceptable provided redistribution of forces is possible. See also DNVGL-RP-C208.

Calculation methods for accidental loads may be found in DNVGL-RP-C204.

Guidance note:

The capability of the structure to redistribute loads during and after accidents should be considered at the design stage. Measures to obtain adequate ductility and redistribution are:

— Select materials with sufficient toughness for the actual service temperature and thickness of structural members.
— Have the strength of connections of primary members exceed the strength of the members themself.
— Provide redundancy in the structure, so that alternate load redistribution paths may be developed.
— Avoid dependency on energy absorption in slender members with a non-ductile post buckling behaviour.
— Avoid pronounced weak sections and abrupt changes in strength or stiffness.

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

SECTION 7 SERVICEABILITY LIMIT STATES

1 General

1.1 General

Serviceability limit states for offshore steel structures are associated with:

— deflections which may prevent the intended operation of equipment
— deflections which may be detrimental to finishes or non-structural elements
— vibrations which may cause discomfort to personnel
— deformations and deflections which may spoil the aesthetic appearance of the structure.

1.2 Deflection criteria

1.2.1 For calculations in the serviceability limit states γ_M = 1.0.

1.2.2 Limiting values for vertical deflections should be given in the design brief. In lieu of such deflection criteria limiting values given in Table 1 may be used.

Table 1 Limiting values for vertical deflections

Condition	*Limit for δ_{max}*	*Limit for δ_2*
Deck beams	$\frac{L}{200}$	$\frac{L}{300}$
Deck beams supporting plaster or other brittle finish or non- flexible partitions	$\frac{L}{250}$	$\frac{L}{350}$
L is the span of the beam. For cantilever beams *L* is twice the projecting length of the cantilever.		

1.2.3 The maximum vertical deflection is:

$$\delta_{max} = \delta_1 + \delta_2 - \delta_0$$

where:

δ_{max}	=	the sagging in the final state relative to the straight line joining the supports
δ_0	=	the pre-camber
δ_1	=	the variation of the deflection of the beam due to the permanent loads immediately after loading
δ_2	=	the variation of the deflection of the beam due to the variable loading plus any time dependent deformations due to the permanent load.

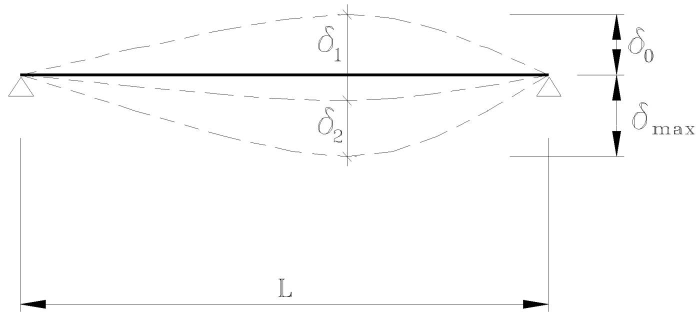


Figure 1 Definitions of vertical deflections

1.2.4 Shear lag effects shall be considered for beams with wide flanges.

1.3 Out of plane deflection of local plates

Check of serviceability limit states for slender plates related to out of plane deflection may be omitted if the smallest span of the plate is less than 150 times the plate thickness.

SECTION 8 WELD CONNECTIONS

1 General

The requirements in this section are related to types and size of welds.

2 Types of welded steel joints

2.1 Butt joints

All types of butt joints should be welded from both sides. Before welding is carried out from the second side, unsound weld metal shall be removed at the root by a suitable method.

Guidance note:

Butt welding from one side against backing (permanent or ceramic) or without backing, may be accepted based on special consideration. One side butt weld shall anyhow not be used inside tanks. Permanent backing or no backing may require special emphasis w.r.t fatigue.

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

2.2 Tee or cross joints

2.2.1 The connection of a plate abutting on another plate may be made as indicated in Figure 1.

2.2.2 The throat thickness of the weld shall always be measured as the normal to the weld surface, as indicated in Figure 1 d).

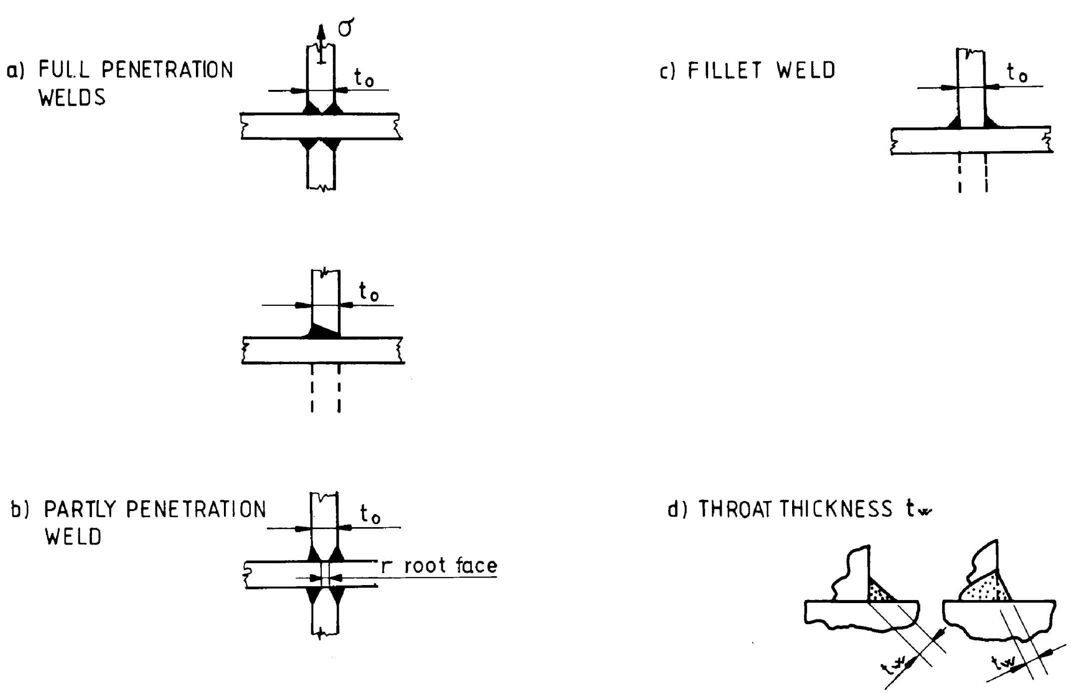


Figure 1 Tee and cross joints

2.2.3 The type of connection shall be adopted as follows:

2.2.3.1 Full penetration weld

— Important cross connections in structures exposed to high stress, especially dynamic, e.g. for special areas and fatigue utilised primary structure.
— All welds with abutting plate panels forming boundaries to open sea.
— All external welds in way of opening to sea e.g. pipes, sea chests or tee-joints.

2.2.3.2 Partly penetration weld

Connections where the static stress level is high. Acceptable also for dynamically stressed connections, provided the equivalent stress is acceptable, see [3.3].

2.2.3.3 Fillet weld

Connections where stresses in the weld are mainly shear, or direct stresses are moderate and mainly static, or dynamic stresses in the abutting plate are small.

2.2.4 Double continuous welds are required in the following connections, irrespective of the stress level:

— oil-tight and watertight connections
— connections at supports and ends of girders, stiffeners and pillars
— connections in foundations and supporting structures for machinery
— connections in rudders, except where access difficulties necessitate slot welds.

2.2.5 Intermittent fillet welds may be used in the connection of girder and stiffener webs to plate and girder flange plate, respectively, where the connection is moderately stressed. See Figure 2, the various types of intermittent welds are as follows:

— chain weld
— staggered weld
— scallop weld (closed).

2.2.6 Where intermittent welds are accepted, scallop welds shall be used in tanks for water ballast or fresh water. Chain and staggered welds may be used in dry spaces and tanks arranged for fuel oil only.

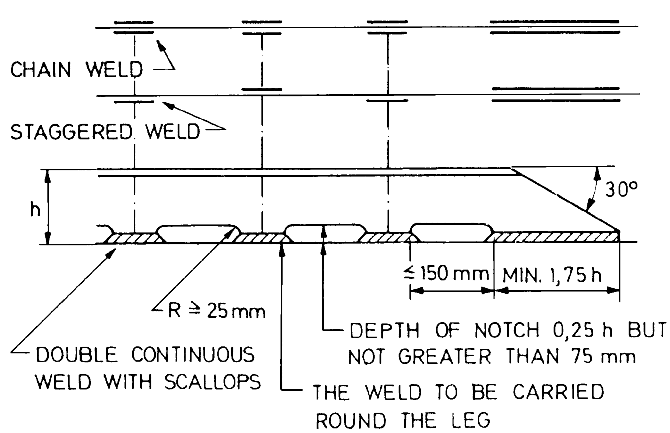


Figure 2 Intermittent welds

2.3 Slot welds

Slot weld, see Figure 3, may be used for connection of plating to internal webs, where access for welding is not practicable, e.g. rudders. The length of slots and distance between slots shall be considered in view of the required size of welding.

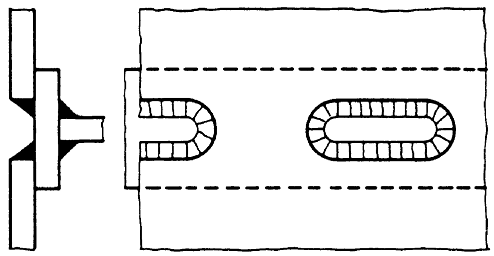


Figure 3 Slot welds

2.4 Lap joint

Lap joint as indicated in Figure 4 may be used in end connections of stiffeners. Lap joints should be avoided in connections with dynamic stresses.

Figure 4 Lap joint

3 Weld size

3.1 General

3.1.1 The material factors γ_{Mw} for welded connections are given in Table 1.

Table 1 Material factors γ_{Mw} for welded connections

Limit states	*Material factor*
ULS	1.3
ALS	1.0

3.1.2 If the yield stress of the weld deposit is higher than that of the base metal, the size of ordinary fillet weld connections may be reduced as indicated in [3.1.4].

3.1.3 Welding consumables used for welding of normal steel and some high strength steels are assumed to give weld deposits with characteristic yield stress σ_{fw} as indicated in Table 2. If welding consumables with deposits of lower yield stress than specified in Table 2 are used, the applied yield strength shall be clearly stated on drawings and in design reports.

Guidance note:

For requirements related to welding consumables see DNVGL-OS-C401 Ch.2 Sec.4, DNVGL-OS-C401 Ch.2 Sec.5 and DNVGL-OS-C401 Ch.3 Sec.1.

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

3.1.4 The size of some weld connections may be reduced:

— corresponding to the strength of the weld metal f_w:

$$f_w = \left(\frac{\sigma_{fw}}{235}\right)^{0,75} \quad or$$

— corresponding to the strength ratio value f_r, base metal to weld metal:

$$f_r = \left(\frac{f_y}{\sigma_{fw}}\right)^{0,75} \quad minimum\ 0.75$$

where:

f_y = characteristic yield stress of base material, abutting plate (N/mm^2)
σ_{fw} = characteristic yield stress of weld deposit (N/mm^2).

Ordinary values for f_w and f_r for normal strength and high strength steels are given in Table 2. When deep penetrating welding processes are applied, the required throat thicknesses may be reduced by 15% provided that sufficient weld penetration is demonstrated.

Table 2 Strength ratios, f_w and f_r

Base metal		*Weld deposit*	*Strength ratios*	
Strength group	*Designation*	*Yield stress, σ_{fw} (N/mm^2)*	*Weld metal* $f_w = \left(\frac{\sigma_{fw}}{235}\right)^{0,75}$	*Base metal/weld metal* $f_r = \left(\frac{f_y}{\sigma_{fw}}\right)^{0,75}$
Normal strength steels	VL NS	355	1.36	0.75
High strength steels	VL 27S	375	1.42	0.75
	VL 32	375	1.42	0.88
	VL 36	375	1.42	0.96
	VL 40	390	1.46	1.00

3.2 Fillet welds

3.2.1 Where the connection of girder and stiffener webs and plate panel or girder flange plate, respectively, are mainly shear stressed, fillet welds as specified in [3.2.2] to [3.2.4] should be adopted.

3.2.2 Unless otherwise calculated, the throat thickness of double continuous fillet welds should not be less than:

$$t_w = 0,43 f_r t_0 \ \ (mm),\ minimum\ 3\ mm$$

where:

f_r = strength ratio as defined in [3.1.4]

t_0 = net thickness (mm) of abutting plate. For stiffeners and for girders within 60% of the middle of span, t_0 need normally not be taken greater than 11 mm, however, in no case less than 0.5 times the net thickness of the web.

3.2.3 The throat thickness of intermittent welds may be as required in [3.2.2] for double continuous welds provided the welded length is not less than:

— 50% of total length for connections in tanks
— 35% of total length for connections elsewhere.

Double continuous welds shall be adopted at stiffener ends when necessary due to bracketed end connections

3.2.4 For intermittent welds, the throat thickness shall not exceed:

— for chain welds and scallop welds:

$$t_w = 0{,}6 f_r t_0 \ \ (mm)$$

where:

t_0 = net thickness abutting plate.

— for staggered welds:

$$t_w = 0{,}75 f_r t_0 \ \ (mm)$$

If the calculated throat thickness exceeds that given in one of the equations above, the considered weld length shall be increased correspondingly.

3.3 Partly penetration welds and fillet welds in cross connections subject to high stresses

3.3.1 In structural parts where dynamic stresses or high static tensile stresses act through an intermediate plate, see Figure 1, penetration welds or increased fillet welds shall be used.

3.3.2 When the abutting plate carries dynamic stresses, the connection shall fulfil the requirements with respect to fatigue, see Sec.5.

3.3.3 When the abutting plate carries design tensile stresses higher than 120 N/mm^2, the throat thickness of a double continuous weld shall not be less than:

$$t_w = \frac{1{,}36}{f_w}\left[0{,}2 + \left(\frac{\sigma_d}{320} - 0{,}25\right)\frac{r}{t_0}\right]t_0 \ \ (mm)$$

minimum 3 mm.

where:

f_w = strength ratio as defined in [3.1.4]

σ_d = calculated maximum design tensile stress in abutting plate (N/mm^2)

r = root face (mm), see Figure 1

t_0 = net thickness (mm) of abutting plate.

3.4 Connections of stiffeners to girders and bulkheads etc.

3.4.1 Stiffeners may be connected to the web plate of girders in the following ways:

— welded directly to the web plate on one or both sides of the stiffener
— connected by single- or double-sided lugs
— with stiffener or bracket welded on top of frame
— a combination of the ways listed above.

In locations where large shear forces are transferred from the stiffener to the girder web plate, a double-sided connection or stiffening should be required. A double-sided connection may be taken into account when calculating the effective web area.

3.4.2 Various standard types of connections between stiffeners and girders are shown in Figure 5.

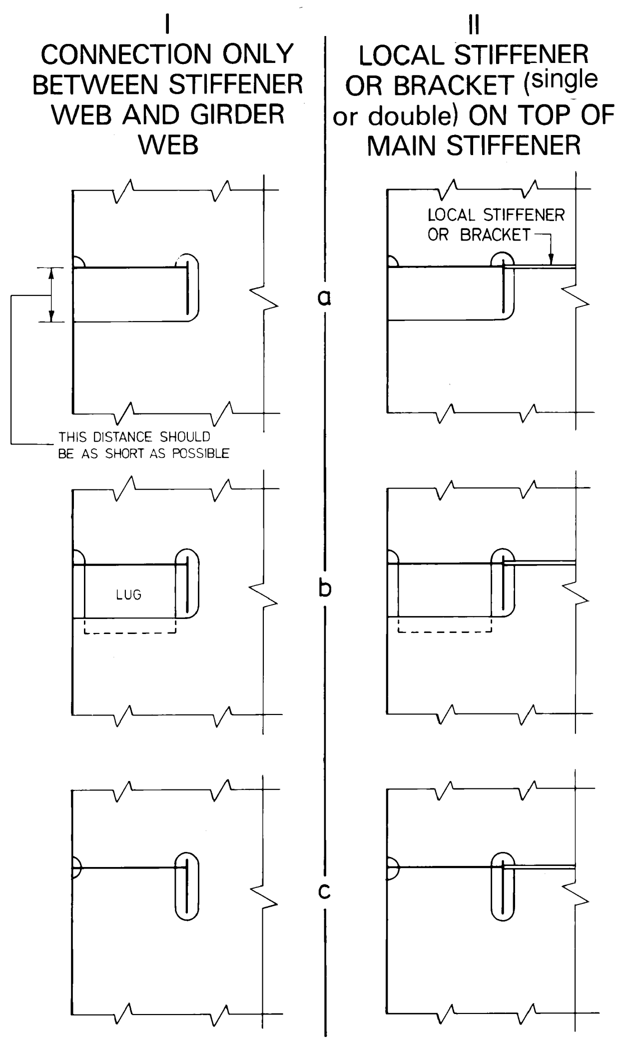


Figure 5 Connections of stiffeners

3.4.3 Connection lugs should have a thickness not less than 75% of the web plate thickness.

3.4.4 The total connection area (parent material) at supports of stiffeners should not be less than:

$$a_0 = \sqrt{3}\,\frac{c}{f_{yd}}10^3(l-0.5s)sp_d \quad (mm^2)$$

where:

c = detail shape factor as given in Table 3

f_{yd} = design yield strength f_y/γ_M.
f_y is the minimum yield stress (N/mm^2) as given in Sec.3 Table 3

l = span of stiffener (m)

s = distance between stiffeners (m)

p_d = design pressure (kN/m^2).

Table 3 Detail shape factor *c*

Type of connection (see Figure 5)	*I Web to web connection only*	*II Stiffener or bracket on top of stiffener*	
		Single-sided	*Double-sided*
a	1.00	1.25	1.00
b	0.90	1.15	0.90
c	0.80	1.00	0.80

3.4.5 Total weld area a shall not be less than:

$$a = f_r a_0 \quad (mm^2)$$

where:

f_r = strength ratio as defined in [3.1.4]

a_0 = connection area (mm^2) as given in [3.4.4].

The throat thickness is not to exceed the maximum for scallop welds given in [3.2.4].

3.4.6 The weld connection between stiffener end and bracket shall principally be designed such that the design shear stresses of the connection correspond to the design resistance.

3.4.7 The weld area of brackets to stiffeners which are carrying longitudinal stresses or which are taking part in the strength of heavy girders etc., shall not be less than the sectional area of the longitudinal.

3.4.8 Brackets shall be connected to bulkhead by a double continuous weld, for heavily stressed connections by a partly or full penetration weld.

3.5 End connections of girders

3.5.1 The weld connection area of bracket to adjoining girders or other structural parts shall be based on the calculated normal and shear stresses. Double continuous welding shall be used. Where large tensile stresses are expected, design according to [3.3] shall be applied.

3.5.2 The end connections of simple girders shall satisfy the requirements for section modulus given for the girder in question. Where the design shear stresses in web plate exceed 90 N/mm^2, double continuous boundary fillet welds should have throat thickness not less than:

$$t_w = \frac{\tau_d}{260 f_w} f_r t_0 \quad (mm)$$

where:

τ_d	=	design shear stress in web plate (N/mm^2)
f_w	=	strength ratio for weld as defined in [3.1.4]
f_r	=	strength ratio as defined in [3.1.4]
t_0	=	net thickness (mm) of web plate.

3.6 Direct calculation of weld connections

3.6.1 The distribution of forces in a welded connection may be calculated on the assumption of either elastic or plastic behaviour.

3.6.2 Residual stresses and stresses not participating in the transfer of load need not be included when checking the resistance of a weld. This applies specifically to the normal stress parallel to the axis of a weld.

3.6.3 Welded connections shall be designed to have adequate deformation capacity.

3.6.4 In joints where plastic hinges may form, the welds shall be designed to provide at least the same design resistance as the weakest of the connected parts.

3.6.5 In other joints where deformation capacity for joint rotation is required due to the possibility of excessive straining, the welds require sufficient strength not to rupture before general yielding in the adjacent parent material.

Guidance note:

In general this will be satisfied if the design resistance of the weld is not less than 80 % of the design resistance of the weakest of the connected parts.

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3.6.6 The design resistance of fillet welds is adequate if, at every point in its length, the resultant of all the forces per unit length transmitted by the weld does not exceed its design resistance.

3.6.7 The design resistance of the fillet weld will be sufficient if both the following conditions are satisfied:

$$\sqrt{\sigma_{\perp d}{}^{2} + 3(\tau_{//d}{}^{2} + \tau_{\perp d}{}^{2})} \leq \frac{f_u}{\beta_w \gamma_{Mw}}$$

$$and \qquad \sigma_{\perp d} \leq \frac{f_u}{\gamma_{Mw}}$$

where:

$\sigma_{\perp d}$	=	normal design stress perpendicular to the throat (including load factors)
$\tau_{\perp d}$	=	shear design stress (in plane of the throat) perpendicular to the axis of the weld
$\tau_{\parallel d}$	=	shear design stress (in plane of the throat) parallel to the axis of the weld, see Figure 6
f_u	=	nominal lowest ultimate tensile strength of base material for the weaker part joined
β_w	=	appropriate correlation factor, see Table 4
γ_{Mw}	=	material factor for welds.

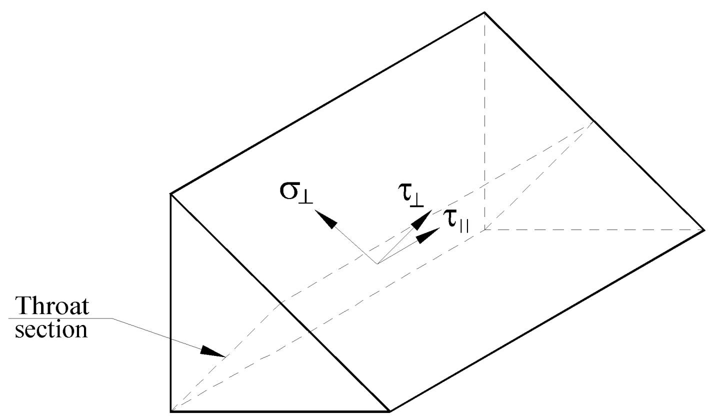

Figure 6 Stresses in fillet weld

Table 4 The correlation factor β_w

Steel grade	*Lowest ultimate tensile strength* f_u	*Correlation factor* β_w
VL NS	400	0.83
VL 27S	400	0.83
VL 32	440	0.86
VL 36	490	0.89
VL 40	510	0.9
VL 420	530	1.0
VL 460	570	1.0

SECTION 9 CORROSION CONTROL

1 Introduction

Corrosion control of structural steel for offshore structures comprises:

— coatings and/or cathodic protection
— use of a corrosion allowance
— inspection/monitoring of corrosion
— control of humidity for internal zones (compartments).

This section gives technical requirements and guidance for the design of corrosion control of structural steel associated with offshore steel structures. The manufacturing/installation of systems for corrosion control and inspection and monitoring of corrosion in operation are covered in DNVGL-OS-C401.

2 Techniques for corrosion control related to environmental zones

2.1 Atmospheric zone

Steel surfaces in the atmospheric zone shall be protected by a coating system (see [4.1]) proven for marine atmospheres by practical experience or relevant testing.

Guidance note:

The atmospheric zone is defined as the areas of a structure above the splash zone (see Sec.9 [2.2.1]) being exposed to sea spray, atmospheric precipitation and/or condensation.

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2.2 Splash zone

2.2.1 Steel surfaces in the splash zone shall be protected by a coating system (see [4.1]) proven for splash zone applications by practical experience or relevant testing. A corrosion allowance should also be considered in combination with a coating system for especially critical structural items.

2.2.2 Steel surfaces in the splash zone, below the mean sea level (MSL) for bottom fixed structures or below the normal operating draught for floating units, shall be designed with cathodic protection in addition to coating.

2.2.3 The splash zone is that part of an installation, which is intermittently exposed to air and immersed in the sea. The zone has special requirements to fatigue for bottom fixed units and floating units that have constant draught.

Guidance note:

Constant draught means that the unit is not designed for changing the draught for inspection and repair for the splash zone and other submerged areas.

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2.2.4 For floating units with constant draught, the extent of the splash zone shall extend 5 m above and 4 m below this draught.

2.2.5 For bottom fixed structures, such as Jack-ups and TLPs, the definitions given in [2.2.5] to [2.2.7] apply. The wave height to be used to determine the upper and lower limits of the splash zone shall be taken as 1/3 of the wave height that has an annual probability of being exceeded of 10^{-2}.

2.2.6 The upper limit of the splash zone (SZ_U) shall be calculated by:

$$SZ_U = U_1 + U_2 + U_3 + U_4 + U_5$$

where:

U_1	=	60% of the wave height defined in [2.2.5]
U_2	=	highest astronomical tide level (HAT)
U_3	=	foundation settlement, if applicable
U_4	=	range of operation draught, if applicable
U_5	=	motion of the structure, if applicable.

The variables (U_i) shall be applied, as relevant, to the structure in question, with a sign leading to the largest or larger value of SZ_U.

2.2.7 The lower limit of the splash zone (SZ_L) shall be calculated by:

$$SZ_L = L_1 + L_2 + L_3 + L_4$$

where:

L_1	=	40% of the wave height defined in [2.2.5]
L_2	=	lowest astronomical tide level (LAT)
L_3	=	range of operating draught, if applicable
L_4	=	motions of the structure, if applicable.

The variables (L_i) shall be applied, as relevant, to the structure in question, with a sign leading to the smallest or smaller value of SZ_L.

2.3 Submerged zone

2.3.1 Steel surfaces in the submerged zone shall have a cathodic protection system. The cathodic protection design shall include current drain to any electrically connected items for which cathodic protection is not considered necessary (e.g. piles). The cathodic protection shall also include the splash zone beneath MSL (for bottom fixed structures) and splash zone beneath normal operating draught (for floating units), see [2.2.2].

Guidance note:

The submerged zone is defined as the zone below the splash zone.

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2.3.2 For certain applications, cathodic protection is only practical in combination with a coating system. Any coating system shall be proven for use in the submerged zone by practical experience or relevant testing demonstrating compatibility with cathodic protection.

Guidance note:

Cathodic protection may cause damage to coatings by blistering or general disbondment (cathodic disbondment).

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2.4 Internal zones

2.4.1 Internal zones exposed to seawater for a main period of time (e.g. ballast tanks) shall be protected by a coating system (see [4.1]) proven for such applications by practical experience or relevant testing. Cathodic protection should be considered for use in combination with coating (see also [2.4.2]).

Guidance note:

Internal zones are defined as tanks, voids and other internal spaces containing a potentially corrosive environment, including seawater.

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

2.4.2 Internal zones that are empty (including those occasionally exposed to seawater for a short duration of time) shall have a coating system and/or corrosion allowance. For internal zones with continuous control of humidity, no further corrosion control is required. Further, no coating is required for zones that do not contain water and that are permanently sealed.

2.4.3 Tanks for fresh water shall have a suitable coating system. Special requirements will apply for coating systems to be used for potable water tanks.

2.4.4 To facilitate inspection, light coloured and hard coatings shall be used for components of internal zones subject to major fatigue forces requiring visual inspection for cracks. Regarding restrictions for use of coatings with high content of aluminium, see [4.1.1].

2.4.5 Only anodes on aluminium or zinc basis shall be used. Due to the risk of hydrogen gas accumulation, anodes of magnesium or impressed current cathodic protection are prohibited for use in tanks.

2.4.6 For cathodic protection of ballast tanks that may become affected by hazardous gas from adjacent tanks for storage of oil or other liquids with flash point less than 60 °C, anodes on zinc basis are preferred. Due to the risk of thermite ignition, any aluminium based anodes shall in no case be installed such that a detached anode could generate an energy of 275 J or higher (i.e. as calculated from anode weight and height above tank top). For the same reason, coatings containing more than 10% aluminium on dry weight basis shall not be used for such tanks.

2.4.7 A corrosion allowance shall be implemented for internal compartments without any corrosion protection (coating and/or cathodic protection) but subject to a potentially corrosive environment such as intermittent exposure to seawater, humid atmosphere or produced/cargo oil. Any corrosion allowance for individual components (e.g. plates, stiffeners and girders) shall be defined taking into account:

— design life
— maintenance philosophy
— steel temperature
— single or double side exposure.

As a minimum, any corrosion allowance (t_k) to be applied as alternative to coating shall be as follows:

— one side unprotected: t_k = 1.0 mm
— two sides unprotected: t_k = 2.0 mm.

In case of permanent exposure to seawater, other corrosive contact, humid atmosphere or produced/cargo oil, the corrosion allowance, t_k, to be applied as alternative to coating shall be as minimum 0.1 mm per year, e.g. 2 mm if the unit design life is 20 years. The corrosion allowance may be reduced based on a regular inspection programme with agreement by the owner.

3 Cathodic protection

3.1 General

3.1.1 Cathodic protection of offshore structures may be effected using galvanic anodes (also referred to as sacrificial anodes) or impressed current from a rectifier. Impressed current is almost invariably used in combination with a coating system.

Guidance note:

The benefits of a coating system (e.g. by reducing weight or friction to seawater flow caused by excessive amounts of anodes) should also be considered for systems based on galvanic anodes.

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3.1.2 Cathodic protection systems in marine environments are typically designed to sustain a protection potential in the range -0.80 V to -1.10 V relative to the Ag/AgCl/seawater reference electrode. More negative potentials may apply in the vicinity of impressed current anodes.

Guidance note:

The use of galvanic anodes based on aluminium and zinc limits the most negative potential to -1.10 V relative to Ag/AgCl/seawater.

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3.1.3 Design of cathodic protection systems for offshore structures shall be carried out according to a recognised standard.

Guidance note:

Recommendations for cathodic protection design may be found in DNVGL-RP-B401.

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

3.1.4 The effect of cathodic protection or exposure to anaerobic environments of excessively strained areas of high and extra high strength steel shall be investigated for the risk of hydrogen induced stress cracking (HISC).

Guidance note:

In this context the cathodic protection potential range in a marine environment as given in [3.1.2] provides sufficient protection against HISC for steels with SMYS up to 550 N/mm^2. For steels in similar areas with SMYS above 550 N/mm^2 the risk of hydrogen embrittlement is reduced by replacing cathodic protection with coating, or qualifying the welding with particular attention to maximum hardness in the weld zone. In the latter case the hardness should be limited to 350 HV (Vickers hardness) during qualification of the welding procedure specification (WPS), and as basis for the hardness readings during welding production testing (WPT). Relevant information should be included in the design and fabrication documentation.

See also DNVGL-OS-C401 Ch.2 Sec.5 [3.3.5.2] andDNVGL-OS-C401 Ch.2 Sec.6 [9.11].

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3.2 Galvanic anode systems

3.2.1 Unless replacement of anodes is allowed for in the design, galvanic anode cathodic protection systems shall have a design life at least equal to that of the offshore installation. For ballast tanks with access for replacement of anodes and any other such applications, the minimum design life should be 5 years.

3.2.2 Anode cores shall be designed to ensure attachment during all phases of installation and operation of the structure. Location of anodes in fatigue sensitive areas shall be avoided.

3.2.3 The documentation of cathodic protection design by galvanic anodes shall contain the following items as a minimum:

— reference to design code and design premises
— calculations of surface areas and cathodic current demand (mean and initial/final) for individual sections of the structure
— calculations of required net anode mass for the applicable sections based on the mean current demands
— calculations of required anode current output per anode and number of anodes for individual sections based on initial/final current demands
— drawings of individual anodes and their location.

3.2.4 Requirements to the manufacturing of anodes (see [3.2.5]) shall be defined during design, e.g. by reference to a standard or in a project specification.

3.2.5 Galvanic anodes shall be manufactured according to a manufacturing procedure specification (to be prepared by manufacturer) defining requirements to the following items as a minimum:

— chemical compositional limits
— anode core material standard and preparation prior to casting
— weight and dimensional tolerances
— inspection and testing
— marking, traceability and documentation.

3.2.6 The needs for a commissioning procedure including measurements of protection potentials at pre-defined locations should be considered during design. As a minimum, recordings of the general protection level shall be performed by lowering a reference electrode from a location above the water level.

3.2.7 Manufacturing and installation of galvanic anodes are addressed in DNVGL-OS-C401 Ch.2 Sec.5.

3.3 Impressed current systems

3.3.1 Impressed current anodes and reference electrodes for control of current output shall be designed with a design life at least equal to that of the offshore installation unless replacement of anodes (and other critical components) during operation is presumed. It is recommended that the design in any case allows for replacement of any defective anodes and reference electrodes (see [3.3.4]) during operation.

3.3.2 Impressed current anodes shall be mounted flush with the object to be protected and shall have a relatively thick non-conducting coating or sheet (dielectric shield) to prevent any negative effects of excessively negative potentials such as disbondment of paint coatings or hydrogen induced damage of the steel. The sizing of the sheet shall be documented during design. Location of impressed current anodes in fatigue sensitive areas shall be avoided.

3.3.3 Impressed current cathodic protection systems shall be designed with a capacity of minimum 1.5 higher than the calculated final current demand of the structure.

Guidance note:

Impressed current cathodic protection provide a more non-uniform current distribution and are more vulnerable to mechanical damage which requires a more conservative design than for galvanic anode systems.

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3.3.4 A system for control of current output based on recordings from fixed reference electrodes located close to and remote from the anodes shall be included in the design. Alarm functions indicating excessive voltage/current loads on anodes, and too negative or too positive protection potential should be provided. A failure mode analysis should be carried out to ensure that any malfunction of the control system will not lead to excessive negative or positive potentials that may damage the structure or any adjacent structures.

3.3.5 Cables from rectifier to anodes and reference electrodes should have steel armour and shall be adequately protected by routing within a dedicated conduit (or internally within the structure, if applicable). Restriction for routing of anode cables in hazardous areas may apply.

3.3.6 The documentation of cathodic protection design by impressed current shall contain the following items as a minimum:

— reference to design code and design premises
— calculations of surface areas and cathodic current demand (mean and initial/final) for individual sections of the structure
— general arrangement drawings showing locations of anodes, anode shields, reference electrodes, cables and rectifiers
— detailed drawings of anodes, reference electrodes and other major components of the system
— documentation of anode and reference electrode performance to justify the specified design life
— documentation of rectifiers and current control system
— documentation of sizing of anode shields
— specification of anode shield materials and application
— commissioning procedure, incl. verification of proper protection range by independent potential measurements
— operational manual, including procedures for replacement of anodes and reference electrodes.

3.3.7 Manufacturing and installation of impressed current cathodic protection systems are addressed in DNVGL-OS-C401 Ch.2 Sec.5.

4 Coating systems

4.1 Specification of coating

4.1.1 Requirements to coatings for corrosion control (including those for any impressed current anode shields) shall be defined during design (e.g. by reference to a standard or in a project specification), including as a minimum:

— coating materials (generic type)
— surface preparation (surface roughness and cleanliness)
— thickness of individual layers
— inspection and testing.

For use of aluminium containing coatings in tanks that may become subject to explosive gas, the aluminium content is limited to maximum 10% on dry film basis.

Guidance note:

A coating technical files (CTF) considering coating work specification, inspection and repair, shall be submitted to the Society before construction. See also MODU Ch.2.12 (2010 edition). It is recommended that supplier specific coating materials are qualified by relevant testing or documented performance in service.

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4.1.2 Coating materials and application of coatings are addressed in DNVGL-OS-C401 Ch.2 Sec.9.

SECTION 10 SOIL FOUNDATION DESIGN

1 General

1.1 Introduction

1.1.1 The requirements in this section apply to pile foundations, gravity type foundations, anchor foundations and stability of sea bottom.

1.1.2 Foundation types not specifically covered by this standard shall be specially considered.

1.1.3 Design of foundations shall be based on site specific information, see [1.2].

1.1.4 The partial coefficient method is the selected method in these standards for foundation design (see Sec.1). The application of this method is documented in this section. Alternative methods or safety checking together with general design principles are given in Sec.1.

1.1.5 In the case where allowable stress methods are used for design, central safety factors shall be agreed upon in each case, with the aim of achieving the same safety level as with the design by the partial coefficient method.

1.1.6 The design of foundations shall consider both the strength and deformations of the foundation structure and of the foundation soils. This section states requirements for:

— foundation soils
— soil reactions upon the foundation structure
— soil-structure interaction.

The foundation structure itself (anchor), including the anchor pad eye, shall be designed for the loads and acceptance criteria specified in DNVGL-OS-E301 Ch.2 Sec.4. See also design requirements in this standard Sec.3 to Sec.9, as relevant for steel anchor design.

1.1.7 A foundation failure mode is defined as the mode in which the foundation reaches any of its limit states. Examples of such failure modes are:

— bearing failure
— sliding
— overturning
— anchor pull-out
— large settlements or displacements.

1.1.8 The definition of limit state categories as given in Sec.1 is valid for foundation design with the exception that failure due to effect of cyclic loading is treated as an ULS limit state, alternatively as an ALS limit state, using load and material coefficients as defined for these limit state categories. The load coefficients shall in this case be applied to all cyclic loads in the design history. Lower load coefficients may be accepted if the total safety level can be demonstrated to be within acceptable limits.

1.1.9 The load coefficients to be used for design related to the different groups of limit states are given in Sec.1. Load coefficients for anchor foundations are given in [5.2].

1.1.10 Material coefficients to be used are specified in [2] to [5]. The characteristic strength of the soil shall be assessed in accordance with [1.3].

1.1.11 Material coefficients shall be applied to soil shear strength as follows:

- For effective stress analysis, the tangent to the characteristic friction angle shall be divided by the material coefficient (γ_M).
- For total shear stress analysis, the characteristic undrained shear strength shall be divided by the material coefficient (γ_M).

For soil resistance to axial pile load, material coefficients shall be applied to the characteristic resistance as described in [3.1.6]. For anchor foundations, material coefficients shall be applied to the characteristic anchor resistance, as described for the respective types of anchors in [5].

1.1.12 Settlements caused by increased stresses in the soil due to structural weight shall be considered for structures with gravity type foundation. In addition, subsidence, e.g. due to reservoir compaction, shall be considered for all types of structures.

1.1.13 Further elaborations on design principles and examples of design solutions for foundation design are given in DNVGL-RP-C212.

1.2 Site investigations

1.2.1 The extent of site investigations and the choice of investigation methods shall take into account the type, size and importance of the structure, uniformity of soil and seabed conditions and the actual type of soil deposits. The area to be covered by site investigations shall account for positioning and installation tolerances.

1.2.2 For anchor foundations the soil stratigraphy and range of soil strength properties shall be assessed within each anchor group or per anchor location, as relevant.

1.2.3 Site investigations shall provide relevant information about the soil to a depth below which possible existence of weak formations will not influence the safety or performance of the structure.

1.2.4 Site investigations are normally to comprise of the following type of investigations:

- site geology survey
- topography survey of the seabed
- geophysical investigations for correlation with borings and in-situ testing
- soil sampling with subsequent laboratory testing
- in-situ tests, e.g. cone penetrations tests.

1.2.5 The site investigations shall provide the following type of geotechnical data for the soil deposits as found relevant for the design:

- data for soil classification and description
- shear strength parameters including parameters to describe the development of excess pore-water pressures
- deformation properties, including consolidation parameters
- permeability
- stiffness and damping parameters for calculating the dynamic behaviour of the structure.

Variations in the vertical, as well as, the horizontal directions shall be documented.

1.2.6 Tests to determine the necessary geotechnical properties shall be carried out in a way that accounts for the actual stress conditions in the soil. The effects of cyclic loading caused by waves, wind and earthquake, as applicable, shall be included.

1.2.7 Testing equipment and procedures shall be adequately documented. Uncertainties in test results shall be described. Where possible, mean and standard deviation of test results shall be given.

1.3 Characteristic properties of soil

1.3.1 The characteristic strength and deformation properties of soil shall be determined for all deposits of importance.

1.3.2 The characteristic value of a soil property shall account for the variability in that property based on an assessment of the soil volume governing for the limit state being considered.

1.3.3 The results of both laboratory tests and in-situ tests shall be evaluated and corrected as relevant on the basis of recognised practice and experience. Such evaluations and corrections shall be documented. In this process account shall be given to possible differences between properties measured in the tests and the soil properties governing the behaviour of the in-situ soil for the limit state in question. Such differences may be due to:

— soil disturbance due to sampling and samples not reconstituted to in-situ stress history
— presence of fissures
— different loading rate between test and limit state in question
— simplified representation in laboratory tests of certain complex load histories
— soil anisotropy effects giving results which are dependent on the type of test.

1.3.4 Possible effects of installation activities on the soil properties should be considered.

1.3.5 The characteristic value of a soil property shall be a cautious estimate of the value affecting the occurrence of the limit state, selected such that the probability of a worse value is low.

1.3.6 A limit state may involve a large volume of soil and it is then governed by the average of the soil property within that volume. The choice of the characteristic value shall take due account for the number and quality of tests within the soil volume involved. Specific care should be made when the limit state is governed by a narrow zone of soil.

1.3.7 The characteristic value shall be selected as a lower value, being less than the most probable value, or an upper value being greater, depending on which is worse for the limit state in question.

Guidance note:

When relevant statistical methods should be used. If such methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state is not greater than 5%.

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1.4 Effects of cyclic loading

1.4.1 The effects of cyclic loading on the soil properties shall be considered in foundation design, where relevant.

1.4.2 Cyclic shear stresses may lead to a gradual increase in pore pressure. Such pore pressure build-up and the accompanying increase in cyclic and permanent shear strains may reduce the shear strength of the soil. These effects shall be accounted for in the assessment of the characteristic shear strength for use in design within the applicable limit state categories.

1.4.3 In the SLS design condition the effects of cyclic loading on the soil's shear modulus shall be corrected for as relevant when dynamic motions, settlements and permanent (long-term) horizontal displacements shall be calculated. See also [4.3].

1.4.4 The effects of wave induced forces on the soil properties shall be investigated for single storms and for several succeeding storms, where relevant.

1.4.5 In seismically active areas, where the structure foundation system may be subjected to earthquake forces, the deteriorating effects of cyclic loading on the soil properties shall be evaluated for the site specific conditions and considered in the design where relevant. See also [1.5].

1.5 Soil-structure interaction

1.5.1 Evaluation of structural load effects shall be based on an integrated analysis of the soil and structure system. The analysis shall be based on realistic assumptions regarding stiffness and damping of both the soil and structural members.

1.5.2 Due consideration shall be given to the effects of adjacent structures, where relevant.

1.5.3 For analysis of the structural response to earthquake vibrations, ground motion characteristics valid at the base of the structure shall be determined. This determination shall be based on ground motion characteristics in free field and on local soil conditions using recognised methods for soil-structure interaction analysis. See Sec.2 [9.1].

2 Stability of seabed

2.1 Slope stability

2.1.1 Risk of slope failure shall be evaluated. Such calculations shall cover:

— natural slopes
— slopes developed during and after installation of the structure
— future anticipated changes of existing slopes
— effect of continuous mudflows
— wave induced soil movements.

The effect of wave loads on the sea bottom shall be included in the evaluation when such loads are unfavourable.

2.1.2 When the structure is located in a seismically active region, the effects of earthquakes on the slope stability shall be included in the analyses.

2.1.3 The safety against slope failure for ULS design shall be analysed using material coefficients (γ_M):

γ_M = 1.2 for effective stress analysis
= 1.3 for total stress analysis.

2.1.4 For ALS design the material coefficients γ_M may be taken equal to 1.0.

2.2 Hydraulic stability

2.2.1 The possibility of failure due to hydrodynamic instability shall be considered where soils susceptible to erosion or softening are present.

2.2.2 An investigation of hydraulic stability shall assess the risk for:

- softening of the soil and consequent reduction of bearing capacity due to hydraulic gradients and seepage forces
- formation of piping channels with accompanying internal erosion in the soil
- surface erosion in local areas under the foundation due to hydraulic pressure variations resulting from environmental loads.

2.2.3 If erosion is likely to reduce the effective foundation area, measures shall be taken to prevent, control and/or monitor such erosion, as relevant, see [2.3].

2.3 Scour and scour protection

2.3.1 The risk for scour around the foundation of a structure shall be taken into account unless it can be demonstrated that the foundation soils will not be subject to scour for the expected range of water particle velocities.

2.3.2 The effect of scour, where relevant, shall be accounted for according to at least one of the following methods:

a) Adequate means for scour protection is placed around the structure as early as possible after installation.
b) The foundation is designed for a condition where all materials, which are not scour resistant are assumed removed.
c) The seabed around the platform is kept under close surveillance and remedial works to prevent further scour are carried out shortly after detection of significant scour.

2.3.3 Scour protection material shall be designed to provide both external and internal stability, i.e. protection against excessive surface erosion of the scour protection material and protection against transportation of soil particles from the underlying natural soil.

3 Design of pile foundations

3.1 General

3.1.1 The load carrying capacity of piles shall be based on strength and deformation properties of the pile material as well as on the ability of the soil to resist pile loads.

3.1.2 In evaluation of soil resistance against pile loads, the following factors shall be amongst those to be considered:

— shear strength characteristics
— deformation properties and in-situ stress conditions of the foundation soil
— method of installation
— geometry and dimensions of pile
— type of loads.

3.1.3 The data bases of existing methods for calculation of soil resistance to axial and lateral pile loads are often not covering all conditions of relevance for offshore piles. This especially relates to size of piles, soil shear strength and type of load. When determining soil resistance to axial and lateral pile loads, extrapolations beyond the data base of a chosen method shall be made with thorough evaluation of all relevant parameters involved.

3.1.4 It shall be demonstrated by a driveability study or equivalent that the selected solution for the pile foundation is feasible with respect to installation of the piles.

3.1.5 Structures with piled foundations shall be assessed with respect to stability for both operation and temporary design conditions, e.g. prior to and during installation of the piles. See Sec.2 for selection of representative loads.

3.1.6 For determination of design soil resistance against axial pile loads in ULS design, a material coefficient γ_M = 1.3 shall be applied to all characteristic values of soil resistance, e.g. to skin friction and tip resistance.

Guidance note:

This material coefficient may be applied to pile foundation of multilegged jacket or template structures. In this application, the design pile load shall be determined from structural analyses where the pile foundation is modelled with elastic stiffness, or non-linear models based on characteristic soil strength.

If the ultimate plastic resistance of the foundation system is analysed by modelling the soil with its design strength and allowing full plastic redistribution until a global foundation failure is reached, higher material coefficients should be used.

For individual piles in a group lower material coefficients may be accepted, as long as the pile group as a whole is designed with the required material coefficient. A pile group in this context shall not include more piles that those supporting one specific leg.

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3.1.7 For pile foundations of structures where there are no or small possibilities for redistribution of loads from one pile to another, or from one group of piles to another group of piles, larger material coefficients than those given in [3.1.6] shall be used. This may for example apply to pile foundations for TLPs or to deep draught floaters. In such cases the material coefficient shall not be taken less than γ_M = 1.7 for ULS design.

3.1.8 For calculation of design lateral resistance according to [3.3], the following material coefficients shall be applied to characteristic soil shear strength parameters for ULS design:

γ_M = 1.2 for effective stress analysis
= 1.3 for total stress analysis.

3.1.9 For ALS and SLS design, the material coefficient γ_M may be taken equal to 1.0.

3.1.10 For conditions where large uncertainties are attached to the determination of characteristic shear strength or characteristic soil resistance, e.g. pile skin friction or tip resistance, larger material factors shall normally be used. Choice of material coefficients shall, in such cases, be in accordance with the determination of characteristic values of shear strength or soil resistance.

3.2 Soil resistance against axial pile loads

3.2.1 Soil resistance against axial pile loads shall be determined by one, or a combination of, the following methods:

— load testing of piles
— semi-empirical pile capacity formulae based on pile load test data.

3.2.2 The soil resistance in compression shall be taken as the sum of accumulated skin friction on the outer pile surface and resistance against pile tip. In case of open-ended pipe piles, the resistance of an internal soil plug shall be taken into account in the calculation of resistance against pile tip. The equivalent tip resistance shall be taken as the lower value of the plugged (gross) tip resistance or the sum of the skin resistance of the internal soil plug and the resistance against the pile tip area. The soil plug may be replaced by a grout plug or equivalent in order to achieve fully plugged tip resistance.

3.2.3 For piles in tension, no resistance from the soil below pile tip shall be accounted for, if the pile tip is in sandy soils.

3.2.4 Effects of cyclic loading shall be accounted for as far as possible. In evaluation of the degradation of resistance, the influence of flexibility of the piles and the anticipated loading history shall be accounted for.

3.2.5 For piles in mainly cohesive soils, the skin friction shall be taken equal to or smaller than the undrained shear strength of undisturbed clay within the actual layer. The degree of reduction depends on the nature and strength of clay, method of installation, time effects, geometry and dimensions of pile, load history and other factors.

3.2.6 The unit tip resistance of piles in mainly cohesive soils may be taken as 9 times the undrained shear strength of the soil near the pile tip.

3.2.7 For piles in mainly cohesionless soils the skin friction may be related to the effective normal stresses against the pile surface by an effective coefficient of friction between the soil and the pile element. It shall be noticed that a limiting value of skin friction may be approached for long piles.

3.2.8 The unit tip resistance of piles in mainly cohesionless soils may be calculated by means of conventional bearing capacity theory, taken into account a limiting value, which may be approached, for long piles.

3.3 Soil resistance against lateral pile loads

3.3.1 When pile penetrations are governed by lateral soil resistance, the design resistance shall be checked within the limit state categories ULS and ALS, using material coefficients as prescribed in [3.1.8].

3.3.2 For analysis of pile stresses and lateral pile head displacement, the lateral soil reaction shall be modelled using characteristic soil strength parameters, with the soil material coefficient γ_M = 1.0. Non-linear response of soil shall be accounted for, including the effects of cyclic loading.

3.4 Group effects

3.4.1 When piles are closely spaced in a group, the effect of overlapping stress zones on the total resistance of the soil shall be considered for axial, as well as, lateral loads on the piles. The increased displacements of the soil volume surrounding the piles due to pile-soil-pile interaction and the effects of these displacements on interaction between structure and pile foundation shall be considered.

3.4.2 In evaluation of pile group effects, due consideration shall be given to factors such as:

— pile spacing
— pile type
— soil strength
— soil density
— pile installation method.

4 Design of gravity foundations

4.1 General

4.1.1 Failure modes within the categories of limit states ULS and ALS shall be considered as described in [4.2].

4.1.2 Failure modes within the SLS, i.e. settlements and displacements, shall be considered as described in [4.2] using material coefficient γ_M = 1.0.

4.2 Stability of foundations

4.2.1 The risk of shear failure below the base of the structure shall be investigated for all gravity type foundations. Such investigations shall cover failure along any potential shear surface with special consideration given to the effect of soft layers and the effect of cyclic loading. The geometry of the foundation base shall be accounted for.

4.2.2 The analyses shall be carried out for fully drained, partially drained or undrained conditions, whatever represents most accurately the actual conditions.

4.2.3 For design within the applicable limit state categories ULS and ALS, the foundation stability shall be evaluated by one of the following methods:

— effective stress stability analysis
— total stress stability analysis.

4.2.4 An effective stress stability analysis shall be based on effective strength parameters of the soil and realistic estimates of the pore water pressures in the soil.

4.2.5 A total stress stability analysis shall be based on total shear strength values determined from tests on representative soil samples subjected to similar stress conditions as the corresponding element in the foundation soil.

4.2.6 Both effective stress and total stress methods shall be based on laboratory shear strength with pore pressure measurements included. The test results should preferably be interpreted by means of stress paths.

4.2.7 Stability analyses by conventional bearing capacity formulae are only acceptable for uniform soil conditions.

4.2.8 For structures where skirts, dowels or similar foundation members transfer loads to the foundation soil, the contributions of these members to the bearing capacity and lateral resistance may be accounted for as relevant. The feasibility of penetrating the skirts shall be adequately documented.

4.2.9 Foundation stability shall be analysed in ULS applying the following material coefficients to the characteristic soil shear strength parameters:

γ_M = 1.2 for effective stress analysis

= 1.3 for total stress analysis.

For ALS design γ_M = 1.0 shall be used.

4.2.10 Effects of cyclic loads shall be included by applying load coefficients in accordance with [1.1.8].

4.2.11 In an effective stress analysis, evaluation of pore pressures shall include:

— initial pore pressure
— build-up of pore pressures due to cyclic load history
— the transient pore pressures through each load cycle
— the effect of dissipation.

4.2.12 The safety against overturning shall be investigated in ULS and ALS.

4.3 Settlements and displacements

4.3.1 For SLS design conditions, analyses of settlements and displacements are, in general, to include calculations of:

— initial consolidation and secondary settlements
— differential settlements
— permanent (long term) horizontal displacements
— dynamic motions.

4.3.2 Displacements of the structure, as well as of its foundation soil, shall be evaluated to provide basis for the design of conductors and other members connected to the structure which are penetrating or resting on the seabed.

4.3.3 Analysis of differential settlements shall account for lateral variations in soil conditions within the foundation area, non-symmetrical weight distributions and possible predominating directions of environmental loads. Differential settlements or tilt due to soil liquefaction shall be considered in seismically active areas.

4.4 Soil reaction on foundation structure

4.4.1 The reactions from the foundation soil shall be accounted for in the design of the supported structure for all design conditions.

4.4.2 The distribution of soil reactions against structural members seated on, or penetrating into the sea bottom, shall be estimated from conservatively assessed distributions of strength and deformation properties of the foundation soil. Possible spatial variation in soil conditions, including uneven seabed topography, shall be considered. The stiffness of the structural members shall be taken into account.

4.4.3 The penetration resistance of dowels and skirts shall be calculated based on a realistic range of soil strength parameters. The structure shall be provided with sufficient capacity to overcome maximum expected penetration resistance in order to reach the required penetration depth.

4.4.4 As the penetration resistance may vary across the foundation site, eccentric penetration forces may be necessary to keep the platform inclination within specified limits.

4.5 Soil modelling for dynamic analysis

4.5.1 Dynamic analysis of a gravity structure shall consider the effects of soil and structure interaction. For homogeneous soil conditions, modelling of the foundation soil using the continuum approach may be used. For more non-homogeneous conditions, modelling by finite element techniques or other recognised methods accounting for non-homogenous conditions shall be performed.

4.5.2 Due account shall be taken of the strain dependency of shear modulus and internal soil damping. Uncertainties in the choice of soil properties shall be reflected in parametric studies to find the influence on response. The parametric studies should include upper and lower boundaries on shear moduli and damping ratios of the soil. Both internal soil damping and radiation damping shall be considered.

4.6 Filling of voids

4.6.1 In order to assure sufficient stability of the structure or to provide a uniform vertical reaction, filling of the voids between the structure and the seabed, e.g. by underbase grouting, may be necessary.

4.6.2 The foundation skirt system and the void filling system shall be designed so that filling pressures do not cause channelling from one compartment to another, or to the seabed outside the periphery of the structure.

4.6.3 The filling material used shall be capable of retaining sufficient strength during the lifetime of the structure considering all relevant forms of deterioration such as:

— chemical
— mechanical
— placement problems such as incomplete mixing and dilution.

5 Design of anchor foundations

5.1 General

5.1.1 This subsection applies to the following types of anchor foundations:

— pile anchors [5.3]
— gravity anchors [5.4]
— suction anchors [5.5]
— fluke anchors [5.6]
— plate anchors [5.7].

5.1.2 The analysis of anchor resistance shall be carried out for the ULS and the ALS, in accordance with the safety requirements given in [5.2]. Due consideration shall be given to the specific aspects of the different anchor types and the current state of knowledge and development.

5.1.3 Determination of anchor resistance may be based on empirical relationships and relevant test data. Due consideration shall be given to the conditions under which these relationships and data are established and the relevance of these conditions with respect to the actual soil conditions, shape and size of anchors and loading conditions.

5.1.4 When clump weight anchors are designed to be lifted off the seabed during extreme loads, due consideration shall be paid to the suction effects that may develop at the clump weight and soil interface during a rapid lift-off. The effect of possible burial during the subsequent set-down shall be considered.

5.2 Safety requirements for anchor foundations

5.2.1 The safety requirements are based on the limit state method of design, where the anchor is defined as a load bearing structure. For geotechnical design of the anchors this method requires that the ULS and ALS categories shall be satisfied by the design.

The ULS is intended to ensure that the anchor can withstand the loads arising in an intact mooring system under extreme environmental conditions. The ALS is intended to ensure that the mooring system retains adequate capacity if one mooring line or anchor should fail for reasons outside the designer's control.

5.2.2 Two consequence classes are considered, both for the ULS and for the ALS, defined as follows:

5.2.2.1 Consequence class 1 (CC1)

Failure is unlikely to lead to unacceptable consequences such as loss of life, collision with an adjacent platform, uncontrolled outflow of oil or gas, capsize or sinking.

5.2.2.2 Consequence class 2 (CC2)

Failure may well lead to unacceptable consequences of these types.

5.2.3 Load coefficients for the two alternative methods to calculate line tension are given in Table 1 and Table 2 for ULS and ALS, respectively. For mooring in deep water (i.e. water depth exceeding 200 m, see DNVGL-OS-E301 Ch.2 Sec.2 [2.1]) a dynamic analysis is required.

Table 1 Load coefficients for ULS

		Time domain analysis		*Frequency domain analysis*)*	
Consequence class	Type of unit	Safety factor on pretension γ_{pret}	Safety factor on env. tension γ_{env}	Safety factor on pretension γ_{pret}	Safety factor on env. tension γ_{env}
1	Permanent	1.20	1.45	1.20	1.80
1	Mobile	1.20	1.35	1.20	1.50
2	Permanent & mobile	1.20	1.90	1.20	2.30

*) The safety factors should be increased by 10% if Rayleigh distributed maxima is assumed for LF motion and WF tension, unless it can be documented that the Rayleigh distribution provide good estimates of the response.

Table 2 Load coefficients for ALS

		Time domain analysis		*Frequency domain analysis*)*	
Consequence class	Type of unit	Safety factor on pretension γ_{pret}	Safety factor on env. tension γ_{env}	Safety factor on pretension γ_{pret}	Safety factor on env. tension γ_{env}
1	Permanent	1.00	1.10	1.00	1.25
1	Mobile	1.00	1.05	1.00	1.10
2	Permanent & mobile	1.00	1.45	1.00	1.70

*) The safety factors should be increased by 10% if Rayleigh distributed maxima is assumed for LF motion and WF tension, unless it can be documented that the Rayleigh distribution provide good estimates of the response.

5.2.4 The design line tension T_d at the touch-down point is the sum of the two line tension components T_{pret} and $T_{C\text{-}env}$ at that point multiplied by their respective load coefficients γ_{pret} and γ_{env}, i.e.:

$$T_d = T_{pret} \cdot \gamma_{pret} + T_{C-env} \cdot \gamma_{env}$$

where:

T_{pret} = mooring line pretension

$T_{C\text{-}env}$ = characteristic environmental line tension induced by mean, low-frequency and wave-frequency loads in the environmental state

5.2.5 Material coefficients for use in combination with the load coefficients in Table 1 and Table 2 are given specifically for the respective types of anchors in [5.3] to [5.7].

5.3 Pile anchors

5.3.1 Pile anchors shall be designed in accordance with the relevant requirements given in [3].

5.3.2 The soil material coefficients to be applied to the resistance of pile anchors shall not be taken less than:

γ_M = 1.3 for ULS consequence class 1 (CC1) and 2 (CC2)

= 1.0 for ALS CC1 and CC2.

See also requirements to tension piles in [3.1.7].

5.4 Gravity anchors

5.4.1 Gravity anchors shall be designed in accordance with the relevant requirements given in [4]. The capacity against uplift of a gravity anchor shall not be taken higher that the submerged mass. However, for anchors supplied with skirts, the contribution from friction along the skirts may be included. In certain cases such anchors may be able to resist cyclic uplift loads by the development of temporary suction within their skirt compartments. In relying on such suction one shall make sure, that there are no possibilities for leakage, e.g. through pipes or leaking valves or channels developed in the soil, that could prevent the development of suction.

5.4.2 The soil material coefficients to be applied to the resistance of gravity anchors shall not be taken less than:

γ_M = 1.3 for ULS consequence class 1 (CC1) and 2 (CC2)

= 1.0 for ALS CC1 and CC2.

5.5 Suction anchors

5.5.1 Suction anchors are vertical cylindrical anchors with open or (normally) closed top, which are installed initially by self-weight penetration followed by application of underpressure (suction) in the closed compartment.

The failure mechanism in the clay around an anchor will depend on various factors, like the load inclination, the anchor depth to diameter ratio, the depth of the load attachment point, the shear strength profile, and whether the anchor has an open or a closed top.

5.5.2 If the load inclination is close to vertical, the anchor will tend to move out of the ground, mainly mobilising the shear strength along the outside skirt wall and the inverse bearing capacity of the soil at skirt tip level. If the anchor has an open top, the inverse bearing capacity will not be mobilised if the inside skirt friction is lower than the inverse bearing capacity at skirt tip level.

5.5.3 If the load inclination is more towards the horizontal, the resistance at the upper part of the anchor will consist of passive and active resistances against the front and back of the anchor, and side shear along the anchor sides. Deeper down, the soil may flow around the anchor in the horizontal plane, or underneath the anchor.

5.5.4 The coupling between vertical and horizontal resistances occurs when the failure mechanism is a combination between vertical and horizontal translation modes. The coupling may reduce the vertical and horizontal resistance components at failure, and the resulting resistance will be smaller than the vector sum of the uncoupled maximum vertical and horizontal resistance. This is illustrated in Figure 1.

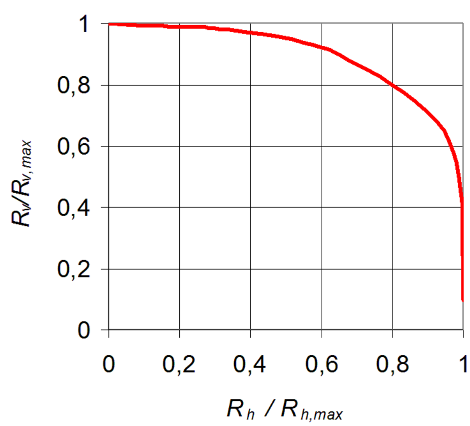

Figure 1 Schematic resistance diagram for suction anchor.

5.5.5 DNV GL recommendations for geotechnical design and installation of suction anchors in clay are provided in DNVGL-RP-E303. The design method outlined in the code makes use of a relatively detailed resistance analysis, and it is concluded that many existing analytical methods will meet the analysis requirements in this code. For details, see DNVGL-RP-E303.

5.5.6 If a less detailed resistance analysis is applied, the designer should be aware of the limitations of the method and make sure that the effects of any simplifications are conservative in comparison with the results from the more advanced methods.

5.5.7 The soil material coefficients to be applied to the resistance of suction anchors shall be:

γ_M = 1.20 for ULS consequence class 1 (CC1) and 2 (CC2)

= 1.20 for ALS CC2, and

= 1.00 for ALS CC1

In the calculation of the anchor resistance, strength anisotropy and the effects of cyclic loading on the undrained shear strength shall be accounted for. The characteristic undrained shear strength shall be taken as the mean value with due account of the quality and complexity of the soil conditions.

5.5.8 Seabed impact landing and subsequent penetration by self-weight shall be addressed in terms of required water evacuation areas to avoid excessive channelling and/or global instability during installation.

5.5.9 Load factors for impact landing, suction to target penetration depth and possible retrieval by means of overpressure shall be taken according to Sec.1 [4.4]. For loads related to permanent removal after design life, the load factor may be taken according to Sec.1 [4.7].

5.5.10 The soil material coefficients to be applied for a potential soil plug failure during suction assisted penetration shall not be taken less than 1.5.

5.6 Fluke anchors

5.6.1 Design of fluke anchors shall be based on recognised principles in geotechnical engineering supplemented by data from tests performed under relevant site and loading conditions.

5.6.2 The penetration resistance of the anchor line shall be taken into considerations where deep penetration is required to mobilise reactions forces.

5.6.3 Fluke anchors shall normally be used only for horizontal and unidirectional load application. However, some uplift may be allowed under certain conditions both during anchor installation and during operating design conditions. The recommended design procedure for fluke anchors is given in the DNVGL-RP-E301.

5.6.4 The required installation load of the fluke anchor shall be determined from the required design resistance of the anchor, allowing for the inclusion of the possible contribution from post installation effects due to soil consolidation and storm induced cyclic loading. For details, see DNVGL-RP-E301. For fluke anchors in sand the same load coefficients as given in [5.2] should be applied, and the target installation load should normally not be taken less than the design load.

5.6.5 Provided that the uncertainty in the load measurements is accounted for and that the target installation tension T_i is reached and verified by reliable measurements the main uncertainty in the anchor resistance lies then in the predicted post-installation effects mentioned above. The soil material coefficient γ_M on this predicted component of the anchor resistance shall then be:

γ_M = 1.3 for ULS consequence class 1 (CC1) and 2 (CC2)
= 1.0 for ALS CC1
= 1.3 for ALS CC2.

5.7 Plate anchors

5.7.1 Design methodologies for plate anchors like drag-in plate anchors, push-in plate anchors, drive-in plate anchors, suction embedment plate anchors, etc. should be established with due consideration of the characteristics of the respective anchor type, how the anchor installation affects the in-place conditions, etc.

5.7.2 Recipes for calculation of characteristic line tension and characteristic anchor resistance are given in DNVGL-RP-E302, together with their partial safety factors for each combination of limit state and consequence class. Requirements for measurements during installation are also provided.

SECTION 11 MISCELLANEOUS

1 Crane pedestals and foundations for lifting appliances

1.1 Application and definitions

1.1.1 This section gives requirements to crane pedestals, foundations for crane pedestals, supporting structure for lifesaving equipment and other lifting appliances. The requirements are applicable when the safe working load (SWL) is greater than 50 kN, or when the maximum overturning moment to the supporting structure is greater than 100 kNm. For davits for survival crafts, man over board-boats and work boats, the requirements are applicable regardless of the SWL or resulting bending moment at hull fixation.

1.1.2 The requirements do not cover the following:

a) Supporting structure for lifting appliances for personnel or passengers, except supporting structure for life saving appliances.
b) Bolted connections between lifting appliances and support structure, as the bolts and their arrangement are considered part of the lifting appliance.

1.1.3 Offshore cranes
For cranes for load handling outside the unit, the requirements are given in DNVGL-ST-0378 Sec.8.

1.1.4 Platform cranes
For cranes for load handling within the unit while at sea or at quay, the requirements are given in DNVGL-ST-0378 Sec.7.

1.1.5 Self-weight
The self-weight is the calculated gross self-weight of the lifting appliance, including the weight of any lifting gear.

1.1.6 Safe working load (SWL)
Safe working load is an international designation for the maximum safe weight, defined by the International Labour Organization (ILO), in tons or kg, the crane is allowed to lift.

1.1.7 Working load (W)
Working load (W) is safe working load (SWL) plus the weight of the lifting gear, e.g. hook block.

1.1.8 Overturning moment
The overturning moment is the maximum bending moment, calculated at the connection of the lifting appliance to the unit, due to the lifting appliance operating at safe working load, taking into account lifting gear, outreach, self-weight, dynamic amplification factors, list and heel, environmental loads as relevant, offlead/sidelead and safety factors.

1.2 Structural categorization

1.2.1 For shipboard/platform and offshore cranes material and inspection categorization for the crane pedestal and the supporting structure is defined in Table 1 below, see Ch.3 for definitions.

Table 1 Material and inspection category

Relevant for	Crane pedestal		Crane pedestal support structure	
	Material categorization	*Inspection categorization*	*Material categorization*	*Inspection categorization*
Shipboard/platform cranes	Primary	IC-II	Primary	IC-II
Offshore Cranes	Primary	IC-II	Special [1)]	IC-I [1)]
1) Extension area for support structure shall be minimum 0.5m in plane from the upper intersection line and 0.5m below this level.				

1.2.2 When a pedestal is not continuous through the deck plating, one of the following applies for the deck plate:

a) Z-quality material shall be used
b) the material may be accepted based on an case-by-case approval, with special attention to the sulphur content which is to satisfy the requirements to Z-quality steels. In addition an ultrasonic test of the plate before welding shall be carried out for the tension exposed areas.

1.3 Structural arrangement

1.3.1 Heavy loaded crane pedestals shall normally be supported by minimum two decks or stringer levels.

1.3.2 Where the pedestal/lifting appliances is welded directly to the deck plate, adequate under deck structure in line with the pedestal/lifting appliances shall be provided.

1.3.3 Full penetration welds shall be used between the aligned structure above and below the deck plate. Structural continuity shall be ensured.

1.4 Design loads and acceptance criteria

1.4.1 The structural strength of the supporting structure (including pedestal) shall be based of a design load consisting of the working load (W) multiplied by the dynamic factor ψ (specified and documented by the crane designer) plus the self-weight. If not otherwise documented, the dynamic factor shall not be taken less than:

– ψ = 1.3 for 50 kN < W ≤ 2500 kN

– ψ = 1.1 for W > 5000 kN.

Linear interpolation to be used for values of W between 2500 kN and 5000 kN. For offshore cranes the design loads for the supporting structure shall be taken as the design loads for the crane multiplied with an additional offshore safety factor SF1 of 1.1. For offshore cranes with W ≤ 2500 kN, where the operator cabin is attached above the slewing bearing, SF1 shall be taken as 1.3. The dynamic factor ψ for lifting appliances fitted with shock absorbers may be specially considered.

1.4.2 Wind and ice load shall be considered as appropriate.

Guidance note:

Standard ice load for North Sea winter conditions may be taken as 5 cm ice deposit for wind and weather exposed surfaces. Wind load shall be based on 1 minute wind speed.

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1.4.3 The crane supplier shall provide the design loads and moments as specified in [1.4.1] at the slewing ring position. In addition the crane supplier shall provide information of the crane fatigue spectre or crane class, and provide reaction force for the boom rest structure as relevant. The crane supplier shall also provide information of:

a) Safe working load (SWL), working load (W) and dynamic factor ψ used as basis for the design cases
b) wind speed used as basis for cranes working with wind
c) ice load when relevant
d) crane self-weigh, S_G, and centre of gravity in stowed condition
e) number of design cycles (class of utilization) and spectrum factor (load spectrum class).

1.4.4 The capacity checks (yield and buckling) of pedestal and supporting structure shall be based on the design loads (incl. dynamic factor) as specified in [1.4.1], with load-factor (γ_f) of 1.25 and material factor (γ_M) of 1.15.

Guidance note:

Using γ_f of 1.25 and γ_M of 1.15 is in line with the WSD format according to DNV GL rules for classification of ships.

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1.4.5 The fatigue damage shall include contribution from operation (when crane is in-use), and stowed condition (crane parked). The total fatigue damage D_{Tot} is found by adding the fatigue damage from operation D_O and the fatigue damage from stowed position D_S.

1.5 Support structure for lifesaving appliances

1.5.1 The structural arrangement shall comply with the requirements given in [1.3].

1.5.2 The structural strength of support structure (including pedestal) shall be based on the safe working load (SWL) times a dynamic factor (ψ) of 2.2, plus the self-weight.

1.5.3 For man-overboard boat davits, the supporting structure shall also be designed to withstand a horizontal towing force.

1.5.4 The capacity checks (yield and buckling) of pedestal and supporting structure shall be based on the working load as specified in [1.5.2], with load-factor (γ_f) of 1.25 and material factor (γ_M) of 1.15.

Guidance note:

This is equivalent with the requirement in the LSA code (LSA code Sec.6.1.1.6) using a safety factor of 4.5 and the material ultimate strength as the allowable limit.

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1.5.5 Support structure exposed to high dynamic loads, typically hull support structure for cantilevered lifeboat davits, shall be appraised for fatigue in stowed condition, D_S. Fatigue contribution related to operation of the davits, D_O, can normally be excluded.

2 Bolt connections

2.1 Bolts and nuts

2.1.1 Bolts and nuts considered essential for structural and operational safety shall conform to a recognised standard, e.g. ISO 898.

2.1.2 Major pressure retaining or structural bolts and nuts with specified min. yield stress above 490 N/mm^2 shall be made of alloy steel, i.e. (% Cr + % Mo + % Ni) ≥ 0.50 and supplied in the quenched and tempered condition.

2.1.3 For general service, the specified tensile properties shall not exceed ISO 898 property class 10.9 when the equipment is in atmospheric environment. For equipment submerged in seawater, the tensile properties shall not exceed property class 8.8 or equivalent.

Guidance note:

For bolted joints to be part of equipment designed for sulphide stress cracking service, lower tensile properties than for property class 8.8 may be necessary in order to comply with NACE MR0175.

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2.2 Slip resistant bolt connections

2.2.1 The requirements in this section give the slip capacity of pre-tensioned bolt connections with high-strength bolts.

2.2.2 A high strength bolt is defined as bolts that have ultimate tensile strength larger than 800 N/mm^2 with yield strength set as minimum 80% of ultimate tensile strength.

2.2.3 The bolt shall be pre-tensioned in accordance with international recognised standards. Procedures for measurement and maintenance of the bolt tension shall be established.

2.2.4 The design slip resistance R_d may be specified equal or higher than the design loads F_d:

$$R_d \geq F_d$$

2.2.5 In addition, the slip resistant connection shall have the capacity to withstand ULS and ALS loads as a bearing bolt connection. The capacity of a bolted connection may be determined according to international recognised standards which give equivalent level of safety such as Eurocode 3.

2.2.6 The design slip resistance of a preloaded high-strength bolt shall be taken as:

$$R_d = \frac{k_s n \mu}{\gamma_{Ms}} F_{pd}$$

where:

k_s = hole clearance factor
= 1.00 for standard clearances in the direction of the force
= 0.85 for oversized holes
= 0.70 for long slotted holes in the direction of the force

n	=	number of friction interfaces
μ	=	friction coefficient
γ_{Ms}	=	1.25 for standard clearances in the direction of the force
	=	1.4 for oversize holes or long slotted holes in the direction of the force
	=	1.1 for design shear forces with load factor 1.0
F_{pd}	=	design preloading force.

2.2.7 For high strength bolts, the controlled design pre-tensioning force in the bolts used in slip resistant connections are:

$$F_{pd} = 0.7 f_{ub} A_s$$

where:

f_{ub}	=	ultimate tensile strength of the bolt
A_s	=	tensile stress area of the bolt (net area in the threaded part of the bolt).

2.2.8 The design value of the friction coefficient μ is dependent on the specified class of surface treatment. The value of μ shall be taken according to Table 4.

Table 2 Friction coefficient μ

Surface category	μ
A	0.5
B	0.4
C	0.3
D	0.2

2.2.9 The classification of any surface treatment shall be based on tests or specimens representative of the surfaces used in the structure, see also DNVGL-OS-C401 Ch.2 Sec.10.

2.2.10 The surface treatments given in Table 3 may be categorised without further testing.

Table 3 Surface treatment

Surface category	*Surface treatment*
A	Surfaces blasted with shot or grit: - with any loose rust removed, no pitting - spray metallised with aluminium - spray metallised with a zinc-based coating certified to prove a slip factor of no less than 0.5.
B	Surfaces blasted with shot or grit, and painted with an alkali-zinc silicate paint to produce a coating thickness of 50 to 80 μm.
C	Surfaces cleaned by wire brushing or flame cleaning, with any loose rust removed.
D	Surfaces not treated.

2.2.11 Normal clearance for fitted bolts shall be assumed if not otherwise specified. The clearances are defined in Table 4.

Table 4 Clearances in bolt holes

Clearance type	*Clearance, in mm*	*Bolt diameter d (maximum), in mm*
Standard	1	12 and 14
	2	16 to 24
	3	27 and larger bolts
Oversized	3	12
	4	14 to 22
	6	24
	8	27

2.2.12 Oversized holes in the outer ply of a slip resistant connection shall be covered by hardened washers.

2.2.13 The nominal sizes of short slotted holes for slip resistant connections shall not be greater than given in Table 5.

Table 5 Short slotted holes

Maximum size, in mm	*Bolt diameter d (maximum), in mm*
(d + 1) by (d + 4)	12 and 14
(d + 2) by (d + 6)	16 to 22
(d + 2) by (d + 8)	24
(d + 3) by (d + 10)	27 and larger

2.2.14 The nominal sizes of long slotted holes for slip resistant connections shall not be greater than given in Table 6.

Table 6 Long slotted holes

Maximum size, in mm	*Bolt diameter d (maximum), in mm*
(d + 1) by 2.5 d	12 and 14
(d + 2) by 2.5 d	16 to 24
(d + 3) by 2.5 d	27 and larger

2.2.15 Long slots in an outer ply shall be covered by cover plates of appropriate dimensions and thickness. The holes in the cover plate shall not be larger than standard holes.

CHAPTER 3 CLASSIFICATION AND CERTIFICATION

SECTION 1 CLASSIFICATION

1 General

1.1 Introduction

1.1.1 As well as representing DNV GL's recommendations on safe engineering practice for general use by the offshore industry, the offshore standards also provide the technical basis for DNV GL classification, certification and verification services.

1.1.2 This chapter identifies the specific documentation, certification and surveying requirements to be applied when using this standard for certification and classification purposes.

1.1.3 A complete description of principles, procedures, applicable class notations and technical basis for offshore classification is given by the applicable rules for classification of offshore units as listed in Table 1.

Table 1 DNV GL rules for classification - Offshore units

Reference	*Title*
DNVGL-RU-OU-0101	Offshore drilling and support units
DNVGL-RU-OU-0102	Floating production, storage and loading units
DNVGL-RU-OU-0103	Floating LNG/LPG production, storage and loading units
DNVGL-RU-OU-0104	Self-elevating units

1.2 Application

1.2.1 It is assumed that the units will comply with the requirement for retention of class as defined in the above listed rule books.

1.2.2 Where codes and standards call for the extent of critical inspections and tests to be agreed between contractor or manufacturer and client, the resulting extent shall be agreed with DNV GL.

1.2.3 DNV GL may accept alternative solutions found to represent an overall safety level equivalent to that stated in the requirements of this standard.

1.2.4 Any deviations, exceptions and modifications to the design codes and standards given as recognised reference codes shall be approved by DNV GL.

1.2.5 In case of classification, the limit states ULS, FLS, and ALS apply. Technical requirements given in Ch.2 Sec.7 related to serviceability limit states (SLS), are not required to be fulfilled as part of classification.

1.3 Documentation

Documentation for classification shall be in accordance with the NPS DocReq (DNV GL Nauticus production system for documentation requirements) and DNVGL-RU-SHIP Pt.1 Ch.3.

APPENDIX A CROSS SECTIONAL TYPES

1 Cross sectional types

1.1 General

1.1.1 Cross sections of beams are divided into different types dependent of their ability to develop plastic hinges as given in Table 1.

Table 1 Cross sectional types

I	Cross sections that can form a plastic hinge with the rotation capacity required for plastic analysis
II	Cross sections that can develop their plastic moment resistance, but have limited rotation capacity
III	Cross sections where the calculated stress in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance
IV	Cross sections where it is necessary to make explicit allowances for the effects of local buckling when determining their moment resistance or compression resistance

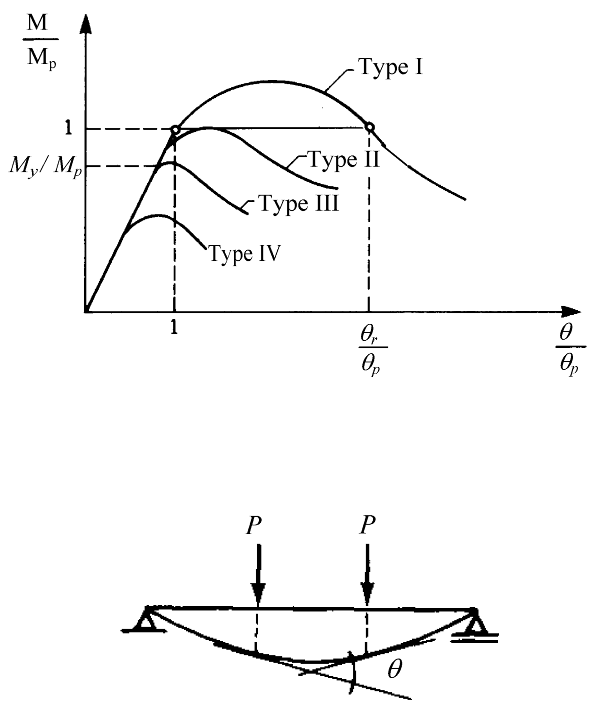


Figure 1 Relation between moment M and plastic moment resistance M_p, and rotation θ for cross sectional types. M_y is elastic moment resistance

1.1.2 The categorisation of cross sections depends on the proportions of each of its compression elements, see Table 3.

1.1.3 Compression elements include every element of a cross section which is either totally or partially in compression, due to axial force or bending moment, under the load combination considered.

1.1.4 The various compression elements in a cross section such as web or flange, can be in different classes.

1.1.5 The selection of cross sectional type is normally quoted by the highest or less favourable type of its compression elements.

1.2 Cross section requirements for plastic analysis

1.2.1 At plastic hinge locations, the cross section of the member which contains the plastic hinge shall have an axis of symmetry in the plane of loading.

1.2.2 At plastic hinge locations, the cross section of the member which contains the plastic hinge shall have a rotation capacity not less than the required rotation at that plastic hinge location.

1.3 Cross section requirements when elastic global analysis is used

1.3.1 When elastic global analysis is used, the role of cross section classification shall identify the extent to which the resistance of a cross section is limited by its local buckling resistance.

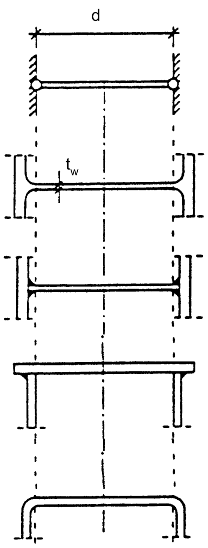
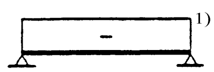
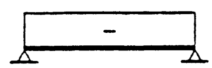
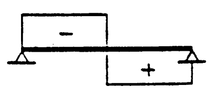
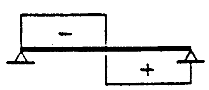
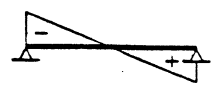
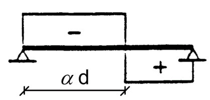
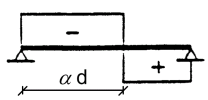
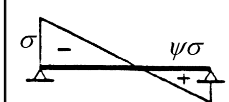
1.3.2 When all the compression elements of a cross section are type III, its resistance may be based on an elastic distribution of stresses across the cross section, limited to the yield strength at the extreme fibres.

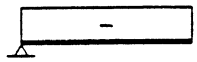
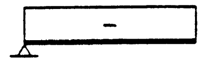
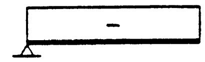
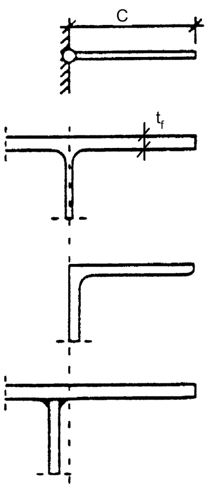
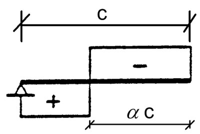
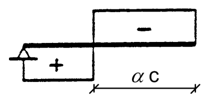
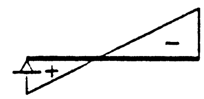
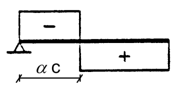
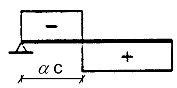
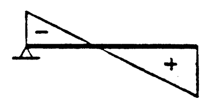
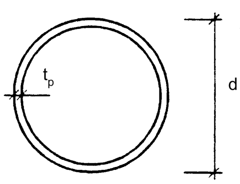
Table 2 Coefficient related to relative strain

VL Steel grade [1)]	ε [2)]
VL NS	1
VL 27S	0.94
VL 32	0.86
VL 36	0.81
VL 40	0.78
VL 420	0.75
VL 460	0.72
VL 500	0.69
VL 550	0.65
VL 620	0.62
VL 690	0.58

VL Steel grade [1]	*ε* [2]
1) The table is not valid for steel with improved weldability. See Ch.2 Sec.3 Table 3 footnote 1). 2) $\varepsilon = \sqrt{\frac{235}{f_y}}$ *where* f_y *is yield strength*	

Table 3 Maximum width to thickness ratios for compression elements

Cross section part	*Type I*	*Type II*	*Type III*
	 $d\ /\ t_w \le 33\ \varepsilon$ [2]	 $d\ /\ t_w \le 38\ \varepsilon$	 $d\ /\ t_w \le 42\ \varepsilon$
	 $d\ /\ t_w \le 72\ \varepsilon$	 $d\ /\ t_w \le 83\ \varepsilon$	 $d\ /\ t_w \le 124\ \varepsilon$
d = h - 3 t_w [3]	 when α > 0.5: $d/t_w \le \frac{396\varepsilon}{13\alpha - 1}$ when α ≤ 0.5: $d/t_w \le \frac{36\varepsilon}{\alpha}$	 when α > 0.5: $d/t_w \le \frac{456\varepsilon}{13\alpha - 1}$ when α ≤ 0.5: $d/t_w \le \frac{41.5\varepsilon}{\alpha}$	 when ψ > -1: $d/t_w \le \frac{126\varepsilon}{2 + \psi}$ when ψ ≤ -1: $d/t_w \le 62\varepsilon(1 - \psi)\sqrt{\lvert\psi\rvert}$

Cross section part	*Type I*	*Type II*	*Type III*
	 *Rolled:* $c/t_f \le 10\varepsilon$ *Welded:* $c/t_f \le 9\varepsilon$	 *Rolled:* $c/t_f \le 11\varepsilon$ *Welded:* $c/t_f \le 10\varepsilon$	 *Rolled:* $c/t_f \le 15\varepsilon$ *Welded:* $c/t_f \le 14\varepsilon$
	Tip in compression  *Rolled:* $c/t_f \le 10\varepsilon/\alpha$ *Welded:* $c/t_f \le 9\varepsilon/\alpha$	Tip in compression  *Rolled:* $c/t_f \le 11\varepsilon/\alpha$ *Welded:* $c/t_f \le 10\varepsilon/\alpha$	Tip in compression  4) *Rolled:* $c/t_f \le 23\varepsilon\sqrt{C}$ *Welded:* $c/t_f \le 21\varepsilon\sqrt{C}$
	Tip in tension  *Rolled:* $c/t_f \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$ *Welded:* $c/t_f \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$	Tip in tension  *Rolled:* $c/t_f \le \frac{11\varepsilon}{\alpha\sqrt{\alpha}}$ *Welded:* $c/t_f \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$	Tip in tension  4) *Rolled:* $c/t_f \le 23\varepsilon\sqrt{C}$ *Welded:* $c/t_f \le 21\varepsilon\sqrt{C}$
 5)	$d/t_p \le 50\,\varepsilon^2$	$d/t_p \le 70\,\varepsilon^2$	$d/t_p \le 90\,\varepsilon^2$

1) Compression negative.
2) ε is defined in Table 2.
3) Valid for rectangular hollow sections (RHS) where h is the height of the profile.
4) C is the buckling coefficient. See DNV-RP-C201 Pt.1 [6.7] or Eurocode 3 (1993-1-5 Table 4.2 denoted k_σ).
5) Valid for axial and bending, not external pressure.

CHANGES – HISTORIC

July 2017 edition

Main changes July 2017

• Ch.2 Sec.3 Structural categorisation, material selection and inspection principles

— Ch.2 Sec.3 [4.2.2]: Extra high strength steel (EHS) included.
— Ch.2 Sec.3 Table 3, Ch.2 Sec.3 Table 4 and Ch.2 Sec.3 Table 5: Tables updated with new material designations according to DNVGL-OS-B101.

April 2016 edition

Main changes April 2016

• Ch.1 Sec.1 Introduction

— [2.1.4]: References in tables updated.
— [3.2] Table 5: Service life removed and replaced with design life and design fatigue life.

• Ch.2 Sec.2 Loads and loads effects

— Table 3: Notes in table updated for variable functional loads.

• Ch.2 Sec.3 Material selection

— Table 3: Material designation updated according to new DNV GL terms.
— Table 5: Strength group column added and material grade naming changed.
— [4.3.8]: Added.

• Ch.2 Sec.4 Ultimate limit states

— [1.4.1]: Added guidance.
— [6.4.4]: Shear area control of stiffeners included.
— Old subsection [8] Slip resistant bolt connections moved to new section Sec.11.

• Ch.2 Sec.6 Accidental limit states

— ALS is general updating in-line with exiting class practice and harmonized against other class object standards.

• Ch.2 Sec.8 Weld connections

— Table 2 and Table 4: Material designation updated according to new DNV GL terms.

• Ch.2 Sec. 9 Corrosion control

— [2.4.7]: Added clarification.

• Ch.2 Sec.11 Miscellaneous

— New section.
— Crane pedestal and foundations for lifting appliance.
— Bolt and nuts moved from DNVGL-OS-C401 and slip resistant bolt connection moved from Sec.4.

July 2015 edition

Main changes July 2015

• General

The revision of this document is part of the DNV GL merger, updating the previous DNV standard into a DNV GL format including updated nomenclature and document reference numbering, e.g.:

— Main class identification 1A1 becomes 1A.
— DNV replaced by DNV GL.
— DNV-RP-A201 to DNVGL-CG-0168. A complete listing with updated reference numbers can be found on DNV GL's homepage on internet.

To complete your understanding, observe that the entire DNV GL update process will be implemented sequentially. Hence, for some of the references, still the legacy DNV documents apply and are explicitly indicated as such, e.g.: Rules for Ships has become DNV Rules for Ships.

About DNV GL

DNV GL is a global quality assurance and risk management company. Driven by our purpose of safeguarding life, property and the environment, we enable our customers to advance the safety and sustainability of their business. We provide classification, technical assurance, software and independent expert advisory services to the maritime, oil & gas, power and renewables industries. We also provide certification, supply chain and data management services to customers across a wide range of industries. Operating in more than 100 countries, our experts are dedicated to helping customers make the world safer, smarter and greener.

SAFER, SMARTER, GREENER