OFFSHORE STANDARD
DNV-OS-C101

DESIGN OF OFFSHORE STEEL STRUCTURES, GENERAL (LRFD METHOD)

OCTOBER 2000

DET NORSKE VERITAS

FOREWORD

DET NORSKE VERITAS (DNV) is an autonomous and independent foundation with the objectives of safeguarding life, property and the environment, at sea and onshore. DNV undertakes classification, certification, and other verification and consultancy services relating to quality of ships, offshore units and installations, and onshore industries worldwide, and carries out research in relation to these functions.

DNV Offshore Codes consist of a three level hierarchy of documents:

— *Offshore Service Specifications*. Provide principles and procedures of DNV classification, certification, verification and consultancy services.
— *Offshore Standards*. Provide technical provisions and acceptance criteria for general use by the offshore industry as well as the technical basis for DNV offshore services.
— *Recommended Practices*. Provide proven technology and sound engineering practice as well as guidance for the higher level Offshore Service Specifications and Offshore Standards.

DNV Offshore Codes are offered within the following areas:

A) Qualification, Quality and Safety Methodology

B) Materials Technology

C) Structures

D) Systems

E) Special Facilities

F) Pipelines and Risers

G) Asset Operation

Amendments April 2002

This Code has been amended, but not reprinted in April 2002. The changes are incorporated in the Web, CD and printable (pdf) versions. The amendments are shown in red colour in the Web and CD versions.

All changes affecting DNV Offshore Codes that have not been reprinted, are published separately in the current *Amendments and Corrections*, issued as a printable (pdf) file.

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SECTION 1
INTRODUCTION

A. General

A 100 Introduction

101 This offshore standard provides principles, technical requirements and guidance on design of offshore structures.

102 DNV-OS-C101 is the general part of the DNV 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 can also be used as a stand-alone document for objects where no object standard exist.

103 When designing a known type of unit, the object standard for the specific type of unit shall be applied. The object standard give references to this standard when appropriate.

104 The standard has been written for general world-wide application. Governmental regulations may include requirements in excess of the the provisions given by this standard depending on the size, type, location and intended service of an offshore unit or installation.

A 200 Objectives

201 The standard specifies general principles and guidelines for the structural design of offshore structures.

202 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
— specify procedures and requirements for offshore structures subject to DNV certification and classification.

203 The general requirements given in this standard may be overruled by specific requirements given in the object standards.

A 300 Scope and application

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

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

303 The standard is applicable to the design of complete structures including substructures, topside structures, vessel hulls and foundations.

304 This standard gives requirements for the following:

— design principles
— selection of material and extent of inspection
— design loads
— load effect analyses
— design of steel structures and connections
— corrosion protection
— foundations.

A 400 Non DNV codes

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

402 Where reference is made to codes other than DNV documents, the valid revision shall be taken as the revision which was current at the date of issue of this standard, unless otherwise noted.

403 When code checks are performed according to other than DNV codes, the resistance or material factors as given in the respective code shall be used.

B. Normative References

B 100 General

101 The standards in Table B1 include provisions, which through reference in this text constitute provisions of this standard.

Table B1 DNV Offshore Service Specifications, Offshore Standards and Rules

Reference	*Title*
DNV-OSS-101	Rules for Classification of Drilling and Support Units
DNV-OSS-102	Rules for Classification of Production and Storage Units
DNV-OS-A101	Safety Principles and Arrangement
DNV-OS-B101	Metallic Materials
DNV-OS-C301	Stability and Watertight Integrity
DNV-OS-C401	Fabrication and Testing of Offshore Structures
DNV-OS-E301	Position Mooring
DNV-OS-E401	Helicopter Decks
	Rules for Classification of Ships
	Rules for Planning and Execution of Marine Operations

C. Informative References

C 100 General

101 The documents in Tables C1, C2 and C3 include acceptable methods for fulfilling the requirements in the standards. See also current DNV List of Publications. Other recognised codes or standards may be applied provided it is shown that they meet or exceed the level of safety of the actual standards.

Table C1 DNV Offshore Standards for structural design

Reference	*Title*
DNV-OS-C102	Structural Design of Offshore Ships
DNV-OS-C103	Structural Design of Column Stabilised Units (LRFD method)
DNV-OS-C104	Structural Design of Self Elevating Units (LRFD method)
DNV-OS-C105	Structural Design of TLP (LRFD method)
DNV-OS-C106	Structural Design of Deep Draught Floating Units

Table C2 DNV Recommended Practices and Classification Notes

Reference	*Title*
DNV-RP-C202	Buckling Strength of Shells
DNV-RP-C203	Fatigue Strength Analysis of Offshore Steel Structures
Classification Note 30.1	Buckling Strength Analysis
Classification Note 30.4	Foundations
Classification Note 30.5	Environmental Conditions and Environmental Loads
Classification Note 30.6	Structural Reliability Analysis of Marine Structures
Classification Note 30.7	Fatigue Assessments of Ship Structures

Table C3 Other references

Reference	*Title*
AISC	LRFD Manual of Steel Construction
API RP 2A LRFD	Planning, Designing, and Constructing Fixed Offshore Platforms - Load and Resistance Factor Design
BS 7910	Guide on methods for assessing the acceptability of flaws in fusion welded structures
Eurocode 3	Design of Steel Structures
ISO 13819-1	Petroleum and natural gas industries – Offshore structures – Part 1: General requirements
NACE TPC	Publication No. 3. The role of bacteria in corrosion of oil field equipment
NORSOK	N-003 Actions and Action Effects
NORSOK	N-004 Design of Steel Structures

D. Definitions

D 100 Verbal forms

101 *Shall:* Indicates a mandatory requirement to be followed for fulfilment or compliance with the present standard. Deviations are not permitted unless formally and rigorously justified, and accepted by all relevant contracting parties.

102 *Should:* Indicates a recommendation that a certain course of action is preferred or particularly suitable. Alternative courses of action are allowable under the standard where agreed between contracting parties but shall be justified and documented.

103 *May:* Indicates a permission, or an option, which is permitted as part of conformance with the standard.

104 *Can:* Requirements with can are conditional and indicate a possibility to the user of the standard.

D 200 Terms

201 *Accidental Limit States (ALS):* Ensures that the structure resists accidental loads and maintain integrity and performance of the structure due to local damage or flooding.

202 *Atmospheric zone:* The external region exposed to atmospheric conditions.

203 *Cathodic protection:* A technique to prevent corrosion of a steel surface by making the surface to be the cathode of an electrochemical cell.

204 *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.

205 *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.

206 *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.

207 *Characteristic value:* The representative value associated with a prescribed probability of not being unfavourably exceeded during some reference period.

208 *Classification Note:* The Classification Notes cover proven technology and solutions which is found to represent good practice by DNV, and which represent one alternative for satisfying the requirements stipulated in the DNV Rules or other codes and standards cited by DNV. The classification notes will in the same manner be applicable for fulfilling the requirements in the DNV offshore standards.

209 *Coating:* Metallic, inorganic or organic material applied to steel surfaces for prevention of corrosion.

210 *Corrosion addition:* Extra steel thickness that may rust away during design life time.

211 *Design brief:* An agreed document where owners requirements in excess of this standard should be given.

212 *Design temperature:* The lowest daily mean temperature to which the structure may be exposed to during installation and operation.

213 *Design value:* The value to be used in the deterministic design procedure, i.e. characteristic value modified by the resistance factor or load factor.

214 *Driving voltage:* The difference between closed circuit anode potential and the protection potential.

215 *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.

216 *Expected value:* The most probable value of a load during a specified time period.

217 *Fatigue:* Degradation of the material caused by cyclic loading.

218 *Fatigue critical:* Structure with calculated fatigue life near the design fatigue life.

219 *Fatigue Limit States (FLS):* Related to the possibility of failure due to the effect of cyclic loading.

220 *Guidance note:* Information in the standards in order to increase the understanding of the requirements.

221 *Hindcasting:* A method using registered meteorological data to reproduce environmental parameters. Mostly used for reproducing wave parameters.

222 *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.

223 *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 state; FLS = fatigue limit state; ALS = accidental limit state; SLS = serviceability limit state.

224 *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.

225 *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.

226 *Lowest mean daily temperature:* The lowest value on the annual mean daily temperature curve for the area in question. For seasonally restricted service the lowest value within the time of operation applies.

227 *Lowest waterline:* Typical light ballast waterline for ships, transit waterline or inspection waterline for other types of units.

228 *Mean:* Statistical mean over observation period.

229 *Non-destructive testing (NDT):* Structural tests and inspection of welds with radiography, ultrasonic or magnetic powder methods.

230 *Object Standard:* The standards listed in Table C1.

231 *Offshore Standard:* The DNV offshore standards are documents which presents the principles and technical requirements for design of offshore structures. The standards are offered as DNV's interpretation of engineering practice for general use by the offshore industry for achieving safe structures.

232 *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.

233 *Operating conditions:* Conditions wherein a unit is on location for purposes of drilling or other similar operations, and combined environmental and operational loadings are within the appropriate design limits established for such operations. The unit may be either afloat or supported on the sea bed, as applicable.

234 *Potential:* The voltage between a submerged metal surface and a reference electrode.

235 *Recommended Practice (RP):* The recommended practice publications cover proven technology and solutions which have been found by DNV to represent good practice, and which represent one alternative for satisfy the requirements stipulated in the DNV offshore standards or other codes and standards cited by DNV.

236 *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 can be achieved for instance by strengthening or introducing alternative load paths.

237 *Reference electrode:* Electrode with stable open-circuit potential used as reference for potential measurements.

238 *Reliability:* The ability of a component or a system to perform its required function without failure during a specified time interval.

239 *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.

240 *Serviceability Limit States (SLS):* Corresponding to the criteria applicable to normal use or durability.

241 *Shakedown:* A linear elastic structural behaviour is established after yielding of the material has occurred.

242 *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.

243 *Specified Minimum Yield Strength (SMYS):* The minimum yield strength prescribed by the specification or standard under which the material is purchased.

244 *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.

245 Splash zone: The external region of the unit which is most frequently exposed to wave action.

246 *Submerged zone:* The part of the installation which is below the splash zone, including buried parts.

247 *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.

248 *Target safety level:* A nominal acceptable probability of structural failure.

249 *Temporary conditions:* An operational condition that may be a design condition. E.g. mating, transit or installation phases.

250 *Tensile strength:* Minimum stress level where strain hardening is at maximum or at rupture.

251 *Transit conditions:* All unit movements from one geographical location to another.

252 *Unit:* is a general term for an offshore installation such as ship shaped, column stabilised, self-elevating, tension leg or deep draught floater.

253 *Utilisation factor:* The fraction of anode material that can be utilised for design purposes.

254 *Verification:* Examination to confirm that an activity, a product or a service is in accordance with specified requirements.

255 *Ultimate Limit States (ULS):* Corresponding to the maximum load carrying resistance.

E. Abbreviations and Symbols

E 100 Abbreviations

101 Abbreviations as shown in Table E1 are used in this standard.

Table E1 Abbreviations

Abbreviation	*In full*
AISC	American Institute of Steel Construction
ALS	accidental limit states
API	American Petroleum Institute
BS	British Standard (issued by British Standard Institute)
CN	classification note
CTOD	crack tip opening displacement
DDF	deep draught floaters
DDF	deep draught floaters
DFF	design fatigue factor
DNV	Det Norske Veritas
EHS	extra high strength
FLS	fatigue limit state

HAT	highest astronomical tide
HISC	hydrogen induced stress cracking
HS	high strength
ISO	international organisation of standardisation
LAT	lowest astronomic tide
LRFD	load and resistance factor design
NACE	National Association of Corrosion Engineers
NDT	non-destructive testing
NS	normal strength
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
TLP	tension leg platform
ULS	ultimate limit states
WSD	working stress design

E 200 Symbols

201 *Latin characters*

α_0	connection area
b	full breadth of plate flange
b_e	effective plate flange width
c	detail shape factor
d	bolt diameter
f	load distribution factor
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	acceleration due to gravity
h	height
h_D	dynamic pressure head due to flow through pipes
h_{pc}	vertical distance from the load point to the position of max filling height
h_s	vertical distance from the load point to the top of the tank
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_r	correction factor for curvature perpendicular to the 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
p_0	valve opening 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 preloading 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.

202 *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
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
σ_{pd1}	design bending stress
σ_{pd2}	design bending stress
τ_d	design shear stress.

203 *Subscripts*

d	design value
k	characteristic value
p	plastic
y	yield.

SECTION 2 DESIGN PRINCIPLES

A. Introduction

A 100 General

101 This section describes design principles and design methods including:

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

102 General design considerations regardless of design method are also given in B101.

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

104 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.

105 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.

A 200 Aim of the design

201 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.

B. General Design Considerations

B 100 General

101 The design of a structural system, its components and details shall, as far as possible, account for the following principles:

- — resistance against relevant mechanical, physical and chemical deterioration is achieved
- — fabrication and construction comply with relevant, recognised techniques and practice
- — inspection, maintenance and repair are possible.

102 Structures and elements thereof, shall possess ductile resistance unless the specified purpose requires otherwise.

103 Structural connections are, in general, to be designed with the aim to minimise stress concentrations and reduce complex stress flow patterns.

104 Fatigue life improvements with methods such as grinding or hammer peening of welds should not provide a measurable increase in the fatigue life at the design stage. The fatigue life should instead be extended by means of modification of structural details. Fatigue life improvements based on mean stress level should not be applied.

105 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 examples where to use plates with proven through thickness properties.

106 Structural elements may be fabricated according to the requirements given in DNV-OS-C401.

C. Limit States

C 100 General

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

102 The following limit states are considered in this standard:

Ultimate limit state (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 state (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.

103 Examples of limit states within each category:

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).

Fatigue limit states (FLS)

- — cumulative damage due to repeated loads.

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).

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.

D. Design by LRFD Method

D 100 General

101 Design by the LRFD method is a design method by which the target component safety level is obtained 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.

102 The target component 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.

D 200 The load and resistance factor design format (LRFD)

201 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.

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

$$F_d = \gamma_f F_k$$

F_d	=	design load
γ_f	=	load factor
F_k	=	characteristic load, see Sec.3.

The load factors and combinations for ULS, ALS, FLS and SLS shall be applied according to 300 to 700.

203 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:

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

S_d	=	design load effect
q	=	load effect function.

204 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:

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

S_{ki}	=	characteristic load effect.

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

206 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).

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

$$R_d = \phi R_k$$

R_k	=	characteristic resistance
ϕ	=	resistance factor.

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

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

γ_M	=	material factor.

208 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.

209 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.

210 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.

D 300 Characteristic load

301 The representative values for the different groups of limit states in the operational design conditions shall be based on Sec.3:

- — 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.

302 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.

D 400 Load factors for ULS

401 For analysis of ULS, two sets of load combinations shall be used when combining design loads as defined in Table D1. 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 D1 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 loadQ = variable functional loadE = environmental loadD = deformation loadFor description of load categories see Sec.3.				

402 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.

403 If a load factor γ_f = 1.0 in combination a) results in higher design load effect, the load factor of 1.0 shall be used.

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

D 500 Load factor for FLS

501 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.

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

D 600 Load factor for SLS

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

D 700 Load factor for ALS

701 The load factors γ_f in the ALS is 1.0.

E. Design Assisted by Testing

E 100 General

101 Design by testing or observation of performance is in general to be supported by analytical design methods.

102 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.

E 200 Full-scale testing and observation of performance of existing structures

201 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.

F. Probability Based Design

F 100 Definition

101 Reliability, or structural safety, is defined as the probability that failure will not occur or that a specified criterion will not be exceeded.

F 200 General

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

202 Acceptable procedures for reliability analyses are documented in the Classification Note 30.6.

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

204 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.

205 Reliability analysis may be updated by utilisation of 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 approval may be revoked.

206 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.

207 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
— Classification Note 30.6.

SECTION 3
LOADS AND LOAD EFFECTS

A. Introduction

A 100 General

101 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.

102 Impact pressure caused by the sea (slamming, bow impact) or by liquid cargoes in partly filled tanks (sloshing) are not covered by this section. Design values are given in the object standards dealing with specific structures.

103 For loads from mooring system, see DNV-OS-E301.

B. Basis for Selection of Characteristic Loads

B 100 General

101 Unless specific exceptions apply, as documented within this standard, the characteristic loads documented in Table B1 and Table B2 shall apply in the temporary and operational design conditions, respectively.

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

Table B1 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 Sec.1.
See Rules for Planing and Execution of Marine Operations.

Table B2 Basis for selection of characteristic loads for operation design conditions

Load category	*Limit states – operation 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 DNV-OS-A101		
Deformation (D)	Expected extreme value				

1) The probability of exceedance applies.

C. Permanent Loads (G)

C 100 General

101 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.

102 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.

D. Variable Functional Loads (Q)

D 100 General

101 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.

102 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
— lifeboats.

103 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.

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

D 200 Variable functional loads on deck areas

201 Variable functional loads on deck areas of the topside structure shall be based on Table D1 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 D1. The following notations are used:

Local design:	e.g. design of plates, stiffeners, beams and brackets
Primary design:	e.g. design of girders and columns
Global design:	e.g design of deck main structure and substructure.

Table D1 Variable functional loads on deck areas

	Local design		*Primary design*	*Global design*
	Distributed loadp (kN/m²)	*Point load,P(kN)*	*Apply factor to distributed load*	*Apply factor to primary design load*
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	4.0	4.0	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 to be added to distributed loads where relevant. (Wheel loads can be considered acting on an area of 300 x 300 mm.)
— Point loads to be applied on an area 100 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. 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.
— Global load cases shall 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.

D 300 Tank pressures

301 The hydrostatic pressures given in 300 and 400 are normative requirements. Other requirements to the hydrostatic pressure in tanks may be given in the object standards.

302 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.

303 Hydrostatic pressures in tanks should be based on a minimum density equal to that of seawater, $\rho = 1025\ \text{kg/m}^3$. 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.

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

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

306 Pumping pressures can 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 can be taken to be the cut-off pressure head h_{pc}.

307 Dynamic pressure heads resulting from filling through pipes by pumping shall be included in the design pressure head.

308 The maximum internal pressure in tanks shall be taken as the largest of pressure p_1 and p_2 given below:

$$p_1 = \rho g(h_{pc} + h_D) \quad (\text{kN/m}^2)$$

ρ	=	density of liquid (kg/m^3)
g	=	9.81 m/s^2
h_{pc}	=	vertical distance (m) from the load point to the position of maximum filling height. If no control devices are used, the pressure height shall be considered to the top of the air pipe. For tanks adjacent to the sea that are situated below the extreme operational draught, hpc should not be taken less than the extreme operational draught h_{pc} should not be taken less than the extreme operational draught
h_D	=	dynamic pressure head due to flow through pipes.

$$p_2 = \rho g h_s + p_0 \quad (\text{kN/m}^2)$$

h_s	=	vertical distance (m) from the load point to the top of the tank
p_0	=	25 kN/m^2 in general. Valve opening pressure when exceeding the general value.

309 Systems installed to limit the pressure to h_{pc} can be taken into account.

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

311 The tank pressures given in this section refer to static pressures only. When hydrostatic pressure is combined with hydrodynamic pressure caused by the motion of the unit, the pressure p_1 shall not be combined with the dynamic pressure (h_D) due to flow resistance in the pipe.

D 400 Miscellaneous loads

401 Railing shall be designed for 1.5 kN/m, acting horizontally on the top of the railing.

E. Environmental Loads (E)

E 100 General

101 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.

102 Practical information regarding environmental loads and conditions are given in Classification Note 30.5.

E 200 Environmental loads for mobile offshore units

201 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 stated in the design basis or the design brief. Unless otherwise stated in the design brief, the North Atlantic scatter diagram should be used in ULS and FLS for unrestricted world wide operation.

E 300 Environmental loads for site specific units

301 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.

302 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.

303 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.

E 400 Determination of characteristic hydrodynamic loads

401 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.

402 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.

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

404 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.

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

406 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.

407 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.

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

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

E 500 Wave loads

501 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.

502 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.

503 Wave loads can be determined according to Classification Note 30.5.

504 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.

505 For slender structures (typically bracings, tendons, risers) where the Morrison equation is applicable, the wave loads can be estimated by careful selection of drag and inertia coefficients, see Classification Note 30.5.

E 600 Wave induced inertia forces

601 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.

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

E 700 Wind loads

701 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.

702 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 Classification Note 30.5.

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

E 800 Earthquake

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

802 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.

803 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.

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

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

806 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.

E 900 Vortex induced oscillations

901 Consideration of loads from vortex shedding on individual elements due to wind, current and waves may be based on Classification Note 30.5. 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.

E 1000 Current

1001 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 Classification Note 30.5.

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E 1100 Tidal effects

1101 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.

1102 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.

Guidance note:

For example, the effects of the highest tide level consistent with the probability of simultaneous occurrence of other extreme environmental conditions should be taken into account in estimating maximum tendon tensions for a TLP.

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E 1200 Marine growth

1201 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.

E 1300 Snow and ice accumulation

1301 Ice accretion from sea spray, snow, rain and air humidity shall be considered, where relevant.

1302 Snow and ice loads may be reduced or neglected if a snow and ice removal procedure is established.

1303 Possible increases of cross-sectional area and changes in surface roughness caused by icing shall be considered, where relevant, when determining wind and hydrodynamic loads.

1304 For buoyant structures the possibility of uneven distribution of snow and ice accretion shall be considered.

E 1400 Direct ice load

1401 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.

1402 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.

E 1500 Water level, settlements and erosion

1501 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.

1502 Uncertainty of measurements and possible erosion shall be considered.

E 1600 Appurtenances and equipment

1601 Hydrodynamic loads on appurtenances (anodes, fenders, strakes etc,) shall be taken into account, when relevant.

F. Combination of Environmental Loads

F 100 General

101 Characteristic values of individual environmental loads are defined by an annual probability of exceedance according to Table F1. The long-term variability of multiple loads is described by a scatter diagram or joint density function including information about direction. Contour curves can then be derived which give combination of environmental parameters which approximately describe the various loads corresponding to the given probability of exceedance.

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

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

104 The load intensities for various types of loads can be selected to correspond to the probabilities of exceedance as given in Table F1.

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

Table F1 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

G. Accidental Loads (A)

G 100 General

101 Accidental loads are loads related to abnormal operations or technical failure. Examples of accidental loads are loads caused by:

— dropped objects
— collision impact
— explosions
— fire
— change of intended pressure difference
— accidental impact from vessel, helicopter or other objects
— unintended change in ballast distribution
— failure of a ballast pipe or unintended flooding of a hull compartment
— failure of mooring lines
— loss of DP system causing loss of heading.

102 Relevant accidental loads should be determined on the basis of an assessment and relevant experiences. With respect to planning, implementation, use and updating of such assessment and generic accidental loads, reference is given to DNV-OS-A101.

103 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 is not to be inferior to the safety level required for the operating design conditions.

H. Deformation Loads (D)

H 100 General

101 Deformation loads are loads caused by inflicted deformations such as:

— temperature loads
— built-in deformations
— settlement of foundations
— the tether pre-tension on a TLP.

H 200 Temperature loads

201 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 born 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.

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

H 300 Settlements and subsidence of sea bed

301 Settlement of the foundations into the sea bed shall be considered.

302 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 service life of the installation, shall be considered.

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

I. Load Effect Analysis

I 100 General

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

— the spatial and temporal nature including:

  — possible non-linearities of the load
  — dynamic character of the response

— the relevant limit states for design check
— the desired accuracy in the relevant design phase.

102 Permanent-, functional-, deformation-, and fire-loads can generally be treated by static methods of analysis. Environmental (wave 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.

103 In general, three frequency bands need to 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).

104 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.

105 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.

106 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.

107 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.2 E200).

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

I 200 Global motion analysis

201 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.

I 300 Load effects in structures and soil or foundation

301 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.

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

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

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

SECTION 4
SELECTION OF MATERIAL AND INSPECTION PRINCIPLES

A. General

A 100

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

B. Design Temperatures

B 100 General

101 The design temperature is a reference temperature used as a criterion for the selection of steel grades. The design temperature shall be based on *lowest daily mean* temperature.

102 In all cases where the service temperature is reduced by localised cryogenic storage or other cooling conditions, such factors shall be taken into account in establishing the minimum design temperatures.

B 200 Floating units

201 The design temperature for floating units shall not exceed the lowest service temperature of the steel as defined for various structural parts.

202 External structures above the lowest waterline shall be designed with service temperatures equal to the *lowest daily mean* temperature for the area(s) where the unit is to operate.

203 Further details regarding design temperature for different structural elements are given in the object standards.

204 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 the lowest average temperature applicable to the relevant actual water depths.

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

B 300 Bottom fixed units

301 For fixed units, materials in structures above the lowest astronomical tide (LAT) shall be designed for service temperatures down to the *lowest daily mean* temperature.

302 Materials in structures below the lowest astronomical tide (LAT) need not to be designed for service temperatures lower than of 0 °C. A higher service temperature may be accepted if adequate supporting data can be presented relative to the *lowest daily mean* temperature applicable for the relevant water depths.

C. Structural Category

C 100 General

101 The purpose of the structural categorisation is to assure adequate material and suitable inspection to avoid brittle fracture. The purpose of inspection is also to remove defects that may grow into fatigue cracks during service 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.

Sharp cracks resulting from fabrication may be found by inspection and repaired. Fatigue cracks may also be discovered during service life by inspection.

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 a suitable fracture toughness for the actual design 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 also DNV-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.

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C 200 Selection of structural category

201 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.

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202 Structural category for selection of materials shall be determined according to principles given in Table C1.

Table C1 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.

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.

C 300 Inspection of welds

301 Requirements for type and extent of inspection are given in DNV-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.

302 The inspection category is by default related to the structural category according to Table C2.

Table C2 Inspection categories	
Inspection category	*Structural category*
I	Special
II	Primary
III	Secondary

303 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.

304 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.

305 Fatigue critical details within structural category *primary* and *secondary* shall be inspected according to requirements in category I.

306 Welds in fatigue critical areas not accessible for inspection and repair during operation shall be inspected according to requirements in category I during construction.

D. Structural Steel

D 100 General

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

102 The requirements in this subsection deal with the selection of various structural steel grades in compliance with the requirements given in DNV-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.

103 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 design temperature, structural category and thickness of the component in question.

104 The material toughness may also be evaluated by fracture mechanics testing in special cases.

105 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 203.

106 Requirements for forging and castings are given in DNV-OS-B101.

D 200 Material designations

201 Structural steel of various strength groups will be referred to as given in Table D1.

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

— steels of normal weldability
— steels of improved weldability.

The two series are intended for the same applications. However, the improved weldability grades have in addition to leaner chemistry and better weldability, extra margins to account for reduced toughness after welding. These grades are also limited to a specified minimum yield stress of 500 N/mm^2.

Table D1 Material designations		
Designation	*Strength group*	*Specified minimum yield stress* f_y (N/mm^2)$^{1)}$
NV	Normal strength steel (NS)	235
NV-27	High strength steel (HS)	265
NV-32		315
NV-36		355
NV-40		390
NV-420	Extra high strength steel (EHS)	420
NV-460		460
NV-500		500
NV-550		550
NV-620		620
NV-690		690
1) For steels of improved weldability the required specified minimum yield stress is reduced for increasing material thickness, see DNV-OS-B101.		

203 Within each strength group different grades, depending upon the required impact toughness properties, are defined. The grades are referred to as A, B, D, E, F or AW, BW, DW, EW for improved weldability grades as shown in Table D2.

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 D2 Applicable steel grades			
Strength group	*Grade*		*Test temperature(°C)*
	Normal weldability	*Improved weldability*	
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
EHS	A	-	0
	D	DW	-20
	E	EW	-40
	F	-	-60
1) Charpy V-notch tests are required for thickness above 25 mm but is subject to agreement between the contracting parties for thickness of 25 mm or less.			

D 300 Selection of structural steel

301 The grade of steel to be used shall in general be related to the design temperature and thickness for the applicable structural category as shown in Table D3.

Table D3 Thickness limitations(mm) of structural steels for different structural categories and design temperatures (°C)					
Structural Category	*Grade*	*≥ 10*	*0*	*-10*	*-20*
Secondary	A	30	30	25	20
	B/BW	60	60	50	40
	D/DW	150	150	100	80
	E/EW	150	150	150	150
	AH/AHW	50	50	40	30
	DH/DHW	100	100	80	60
	EH/EHW	150	150	150	150
	FH	150	150	150	150
	AEH	60	60	50	40
	DEH/DEHW	150	150	100	80
	EEH/EEHW	150	150	150	150
	FEH	150	150	150	150
Primary	A	30	20	10	N.A.
	B/BW	40	30	25	20
	D/DW	60	60	50	40
	E/EW	150	150	100	80
	AH/AHW	25	25	20	15
	DH/DHW	50	50	40	30
	EH/EHW	100	100	80	60
	FH	150	150	150	150
	AEH	30	30	25	20
	DEH/DEHW	60	60	50	40
	EEH/EEHW	150	150	100	80
	FEH	150	150	150	150
Special	D/DW	35	30	25	20
	E/EW	60	60	50	40
	AH/AHW	10	10	N.A.	N.A.
	DH/DHW	25	25	20	15
	EH/EHW	50	50	40	30
	FH	100	100	80	60
	AEH	15	15	10	N.A.
	DEH/DEHW	30	30	25	20
	EEH/EEHW	60	60	50	40
	FEH	150	150	100	80
N.A. = no application					

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

303 Grade of steel to be used for thickness less than 10 mm and/or design temperature above 0 °C will be specially considered in each case.

304 Welded steel plates and sections of thickness exceeding the upper limits for the actual steel grade as given in Table D3 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, e.g. in accordance with BS 7910.

305 For regions subjected to compressive and/or low tensile stresses, consideration may be given to the use of lower steel grades than stated in Table D3.

306 The use of steels with specified minimum yield stress greater than 550 N/mm^2 (NV550) 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.

307 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).

308 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.10.

SECTION 5
ULTIMATE LIMIT STATES

A. General

A 100 General

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

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

103 The structural capacity of all structural components shall be performed. The capacity check shall consider both excessive yielding and buckling.

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

105 The corrosion addition as given in Sec.10 B500 shall not be accounted for in the determination of design resistance.

A 200 Structural analysis

201 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.

202 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 needs to 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.

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

204 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.

205 When plastic analysis and/or plastic capacity checks are used (cross section type I and II, according to Appendix 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.

206 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 Appendix A.

A 300 Ductility

301 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.

302 The following sources for brittle structural behaviour may need to 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.

A 400 Yield check

401 Structural members for which excessive yielding is a possible mode of failure, are to be investigated for yielding.

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

403 Yield checks may be performed based on net sectional properties. For large volume hull structures gross scantlings may be applied.

404 For yield check of welded connections, see Sec.9.

A 500 Buckling check

501 Requirements for the elements of the cross section not fulfilling requirements to cross section type III need to be checked for local buckling.

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

503 The characteristic buckling strength shall be based on the 5^{th} percentile of test results.

504 Initial imperfections and residual stresses in structural members shall be accounted for.

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

Guidance note:
If buckling resistance is calculated in accordance with Classification Note 30.1 or DNV-RP-C202, the maximum imperfections as given in Classification Note 30.1 should not be exceeded.

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B. Flat Plated Structures and Stiffened Panels

B 100 General

101 The material factor γ_M for plated structures is 1.15.

B 200 Yield check

201 Yield check of plating and stiffeners may be performed as given in F.

202 Yield check of girders may be performed as given in G.

B 300 Buckling check

301 The buckling stability of plated structures may be checked according to Classification Note 30.1, Sec. 3.

Guidance note:

When using Classification Note 30.1 for check of buckling, the allowable usage factor should be set to $1/\gamma_M$ and the load effects should be based on factorised loads. For unstiffened flat plate panels, this usage factor may be increased 10% using Classification Note 30.1.

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302 In case the stiffened panel is buckling checked as a stiffener with effective plate width, the plate between the stiffeners need not to be checked separately.

B 400 Capacity checks according to other codes

401 Stiffeners and girders may be designed according to provisions for beams in recognised standards such as Eurocode 3 or AISC LRFD Manual of Steel Construction.

402 Material factors when using Eurocode 3 are given in Table B1.

Table B1 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	γ_{M1}	1.15
1) Symbols according to Eurocode 3.		

Guidance note:

The principles and effects of cross section types are included in the AISC LRFD Manual of Steel Construction.

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403 Plates, stiffeners and girders may be designed according to NORSOK N-004.

C. Shell Structures

C 100 General

101 The buckling stability of shell structures may be checked according to DNV-RP-C202.

102 For interaction between shell buckling and column buckling, DNV-RP-C202 may be used.

103 If DNV-RP-C202 is applied, the material factor for shells shall be in accordance with Table C1.

Table C1 Material factors γ_M for buckling

Type of structure	$\lambda \leq 0.5$	$0.5 < \lambda < 1.0$	$\lambda \geq 1.0$
Girder, beams stiffeners on shells	1.15	1.15	1.15
Shells of single curvature (cylindrical shells, conical shells)	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
	=	$\sqrt{\dfrac{f_y}{\sigma_e}}$
f_y	=	specified minimum yield stress
σ_e	=	elastic buckling stress for the buckling mode under consideration.

D. Tubular Members, Tubular Joints and Conical Transitions

D 100 General

101 Tubular members without external pressure may be checked according to Classification Note 30.1. Tubular members with external pressure and with compact cross sections may be checked according to Classification Note 30.1.

Guidance note:

Compact tubular cross section is in this context defined as when the diameter (D) to thickness (t) ratio satisfy the following criteria:

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

E = modulus of elasticity and
f_y = minimum yield strength

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102 Tubular members with external pressure, tubular joints and conical transitions may be checked according to API RP 2A – LRFD or NORSOK N-004.

103 The material factor γ_M for tubular structures is 1.15.

E. Non-Tubular Beams, Columns and Frames

E 100 General

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

102 Buckling checks may be performed according to Classification Note 30.1.

103 Capacity check may be performed according to recognised standards such as Eurocode 3 or AISC LRFD Manual of Steel Construction.

104 The material factors according to Table B1 shall be used if Eurocode 3 is used for calculation of structural resistance.

F. Special Provisions for Plating and Stiffeners

F 100 Scope

101 The requirements in F will normally give minimum scantlings to plate and stiffened panels with respect to yield. Dimensions and further references with respect to buckling capacity are given in B.

F 200 Minimum thickness

201 The thickness of plates should not to be less than:

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

f_{yd} = design yield strength f_y/γ_M
f_y is the minimum yield stress (N/mm^2) as given in Sec.4 Table D1
t_0 = 7 mm for primary structural elements
= 5 mm for secondary structural elements
γ_M = material factor for steel
= 1.15.

F 300 Bending of plating

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

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

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 $s/l = 1.0$
k_r	=	correction factor for curvature perpendicular to the stiffeners
	=	$(1 - 0.5\ s/r_c)$
r_c	=	radius of curvature (m)
s	=	stiffener spacing (m), measured along the plating
p_d	=	design pressure (kN/m^2) as given in Sec.3
σ_{pd1}	=	design bending stress
	=	$1.3\ (f_{yd} - \sigma_{jd})$, but less than $f_{yd} = f_y / \gamma_M$
σ_{jd}	=	equivalent design stress for global in-plane membrane stress
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.

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F 400 Stiffeners

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

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

l	=	stiffener span (m)
k_m	=	bending moment factor, see Table G1
σ_{pd2}	=	design bending stress
	=	$f_{yd} - \sigma_{jd}$
k_{ps}	=	fixation parameter for stiffeners
	=	1.0 if at least one end is clamped
	=	0.9 if both ends are simply supported.

402 The formula given in 401 shall be regarded as the requirement about an axis parallel to the plating. As an approximation the requirement for standard section modulus for stiffeners at an oblique angle with the plating may be obtained if the formula in 401 is multiplied by the factor:

$$\cos\left(\frac{1}{\alpha}\right)$$

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

403 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 401 is normally to 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.

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G. Special Provisions for Girder and Girder Systems

G 100 Scope

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

102 Dimensions and further references with respect to buckling capacity are given in B.

G 200 Minimum thickness

201 The thickness of web and flange plating is not to be less than given in F200 and F300.

G 300 Bending and shear

301 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 will have to be specially considered.

302 When boundary conditions for individual girders are not predictable due to dependence of adjacent structures, direct calculations according to the procedures given in 700 will be required.

303 The section modulus and web area of the girder shall be taken in accordance with particulars as given in 600 or 700. Structural modelling in connection with direct stress analysis shall be based on the same particulars when applicable.

G 400 Effective flange

401 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 be is determined by the following formula:

$$b_e = C_e b$$

C_e	=	as given in Fig.1 for various numbers of evenly spaced point loads (N_p) on the span
b	=	full breadth of plate flange e.g. span of the stiffeners supported by the girder with effective flange b_e, see also 602.

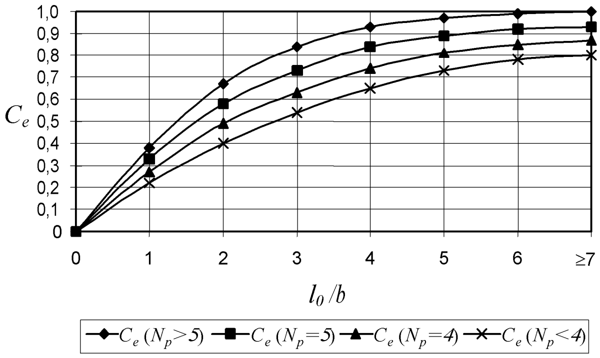


Figure 1
Graphs for the effective flange parameter C

l_0 = distance between points of zero bending moments (m)
= S for simply supported girders
= 0.6 S for girders fixed at both ends

S = girder span as if simply supported, see also 602.

G 500 Effective web

501 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.

G 600 Strength requirements for simple girders

601 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:

— net section modulus according to 602
— net web area according to 603.

602 *Section modulus*

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

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) (m), l_1 and l_2 are the spans of the supported stiffeners, or distance between girders

k_m = bending moment factor k_m-values in accordance with Table G1 may be applied

σ_{pd2} = design bending stress
= $f_{yd} - \sigma_{jd}$

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

603 *Net web area*

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

k_τ = shear force factor k_τ may be in accordance with 604

N_s = number of stiffeners between considered section and nearest support
The N_s-value is in no case to be taken greater than $(N_p+1)/4$.

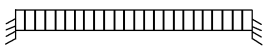
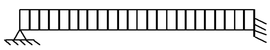
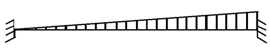
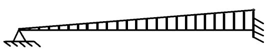
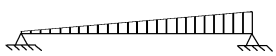
N_p = number of supported stiffeners on the girder span

P_{pd} = average design point load (kN) from stiffeners between considered section and nearest support

τ_p = 0.5 f_{yd} (N/mm^2).

604 The k_m- and k_τ-values referred to in 602 and 603 may be calculated according to general beam theory. In Table G1 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 G1 Values of k_m and k_τ

Load and boundary conditions			*Bending moment and shear force factors*		
Positions			1$k_{m1}k_{\tau1}$	2k_{m2}-	3$k_{m3}k_{\tau3}$
1Support	2Field	3Support			
			120.5	24	120.5
			0.38	14.2	80.63
			0.5	8	0.5
			150.3	23.3	100.7
			0.2	16.8	7.50.8
			0.33	7.8	0.67

G 700 Complex girder system

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

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

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

704 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.

705 The most unfavourable of the loading conditions given in Sec.3 shall be applied.

706 For girders taking part in the overall strength of the unit, stresses due to the design pressures given in Sec.3 shall be combined with relevant overall stresses.

H. Slip Resistant Bolt Connections

H 100 General

101 The requirements in H give the slip capacity of pre-tensioned bolt connections with high-strength bolts.

102 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.

103 The bolt shall be pre-tensioned in accordance with international recognised standards. Procedures for measurement and maintenance of the bolt tension shall be established.

104 The design slip resistance R_d may be specified equal or higher than the design loads F_d.

$$R_d \geq F_d$$

105 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 or AISC LRFD Manual of Steel Construction.

106 The design slip resistance of a preloaded high-strength bolt shall be taken as:

$$R_d = \frac{k_s n \mu}{\gamma_{Ms}} F_{pd}$$

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.

107 For high strength bolts, the controlled design pre-tensioning force in the bolts used in slip resistant connections are:

$$F_{pd} = 0.7 f_{ub} A_s$$

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).

108 The design value of the friction coefficient μ is dependent on the specified class of surface treatment as given in DNV-OS-C401 Sec.7. The value of μ shall be taken according to Table H1.

Table H1 Friction coefficient μ

Surface category	μ
A	0.5
B	0.4
C	0.3
D	0.2

109 The classification of any surface treatment shall be based on tests or specimens representative of the surfaces used in the structure using the procedure set out in DNV-OS-C401.

110 Provided the contact surfaces have been treated in conformity with DNV-OS-C401 Sec.7, the surface treatments given in Table H2 may be categorised without further testing.

Table H2 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 not 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 mm
C	Surfaces cleaned by wire brushing or flame cleaning, with any loose rust removed
D	Surfaces not treated

111 Normal clearance for fitted bolts shall be assumed if not otherwise specified. The clearances are defined in Table H3.

Table H3 Clearances in bolt holes

Clearance type	*Clearance mm*	*Bolt diameter d (maximum) 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

112 Oversized holes in the outer ply of a slip resistant connection shall be covered by hardened washers.

113 The nominal sizes of short slotted holes for slip resistant connections shall not be greater than given in Table H4.

Table H4 Short slotted holes	
Maximum size mm	*Bolt diameter d (maximum) 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

114 The nominal sizes of long slotted holes for slip resistant connections shall not be greater than given in Table H5.

Table H5 Long slotted holes	
Maximum size mm	*Bolt diameter d (maximum) 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

115 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.

SECTION 6
FATIGUE LIMIT STATES

A. General

A 100 General

101 In this standard, requirements are given in relation to fatigue analyses based on fatigue tests and fracture mechanics. Reference is made to DNV-RP-C203 and Classification Note 30.7 for practical details with respect to fatigue design. See also Sec.2 B104.

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

103 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.

104 The S-N curves shall in general be based on a 97.6 % probability of survival.

105 The design fatigue life for structural components should be based on the specified service life of the structure. If a service life is not specified, 20 years should be used.

106 To ensure that the structure will fulfil the intended function, a fatigue assessment shall be carried out for each individual member 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 member of the structure, every welded joint and attachment or other form of stress concentration is potentially a source of fatigue cracking and should be individually considered.

A 200 Design fatigue factors

201 Design fatigue factors (DFF) shall be applied to increase the probability for avoiding fatigue failures.

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

203 DFFs shall be applied to the design fatigue life. The calculated fatigue life shall be longer than the design fatigue life times the DFF.

204 The design requirement can alternatively be expressed as the cumulative damage ratio for the number of load cycles of the defined design fatigue life multiplied with the DFF shall be less or equal to 1.0.

205 The design fatigue factors in Table A1 are valid for units with low consequence of failure and where it can be demonstrated that the structure satisfy the requirement to damaged condition according to the ALS with failure in the actual joint as the defined damage.

Table A1 Design fatigue factors (DFF)

DFF	*Structural element*
1	Internal structure, accessible and not welded directly to the submerged part.
1	External structure, accessible for regular inspection and repair in dry and clean conditions.
2	Internal structure, accessible and welded directly to the submerged part.
2	External structure not accessible for inspection and repair in dry and clean conditions.
3	Non-accessible areas, areas not planned to be accessible for inspection and repair during operation.

Guidance note:

For units inspected during operation according to DNV requirements, the DFF for outer shell should be taken as 1. Units that are planned to be inspected afloat at a sheltered location the DFF for areas above 1 m above lowest inspection waterline should be taken as 1, and below this line the DFF is 2 for the outer shell. Splash zone is defined as non-accessible area.

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.

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206 The design fatigue factors shall be based on special considerations where fatigue failure will entail substantial consequences such as:

- danger of loss of human life, i.e. not compliance with ALS criteria
- significant pollution
- major economical consequences.

Guidance note:

Evaluation of likely crack propagation paths (including direction and growth rate related to the inspection interval), may indicate the use of a different DFF than that which would be selected when the detail is considered in isolation. E.g. where the likely crack propagation indicates that a fatigue failure starting in a non critical area grows such that there might be a substantial consequence of failure, such fatigue sensitive location should itself be deemed to have a substantial consequence of failure.

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207 Welds in joints below 150 m water depth should be assumed inaccessible for in-service inspection.

208 The object standards define the design fatigue factors to be applied for typical structural details.

A 300 Methods for fatigue analysis

301 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.

302 In fatigue critical areas where the fatigue life estimate based on simplified methods is short, a more accurate investigation or a fracture mechanics analysis shall be performed.

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

304 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 7
ACCIDENTAL LIMIT STATES

A. General

A 100 General

101 The ALS shall in principle be assessed for all units. Safety assessment is carried out according to the principles given in DNV-OS-A101.

102 The material factor γ_M for the ALS is 1.0.

103 Structures shall be checked in ALS in two steps:

a) Resistance of the structure against design accidental loads.

b) Post accident resistance of the structure against environmental loads should only be checked when the resistance is reduced by structural damage caused by the design accidental loads.

104 The overall objective of design against accidental loads is to achieve a system where the main safety functions are not impaired by the design accidental loads.

105 The design against accidental loads may be done by direct calculation of the effects imposed by the loads on the structure, or indirectly, by design of the structure as tolerable to accidents. Examples of the latter are compartmentation of floating units which provides sufficient integrity to survive certain collision scenarios without further calculations.

106 The inherent uncertainty of the frequency and magnitude of the accidental loads, as well as the approximate nature of the methods for determination of accidental load effects, shall be recognised. It is therefore essential to apply sound engineering judgement and pragmatic evaluations in the design.

107 If non-linear, dynamic finite element analysis is applied for design, it shall be verified that all local failure mode, e.g. strain rate, local buckling, joint overloading, joint fracture, are accounted for implicitly by the modelling adopted, or else subjected to explicit evaluation.

Typical accidental loads are:

— impact from ship collisions
— impact from dropped objects
— fire
— explosions
— abnormal environmental conditions
— accidental flooding.

108 The different types of accidental loads require different methods and analyses to assess the structural resistance.

SECTION 8
SERVICEABILITY LIMIT STATES

A. General

A 100 General

101 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.

A 200 Deflection criteria

201 For calculations in the serviceability limit states $\gamma_M = 1.0$

202 Limiting values for vertical deflections should be given in the design brief. In lieu of such deflection criteria limiting values given in Table A1 may be used.

Table A1 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.		

203 The maximum vertical deflection is:

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

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

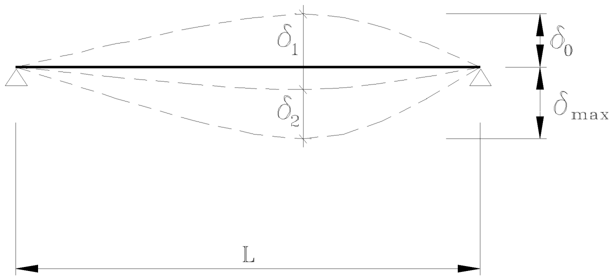

Figure 1
Definitions of vertical deflections

204 Shear lag effects need to be considered for beams with wide flanges.

A 300 Out of plane deflection of local plates

301 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 9 WELD CONNECTIONS

A. General

A 100 General

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

B. Types of Welded Steel Joints

B 100 Butt joints

101 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.

B 200 Tee or cross joints

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

202 The throat thickness of the weld is always to be measured as the normal to the weld surface, as indicated in Fig. 1d.

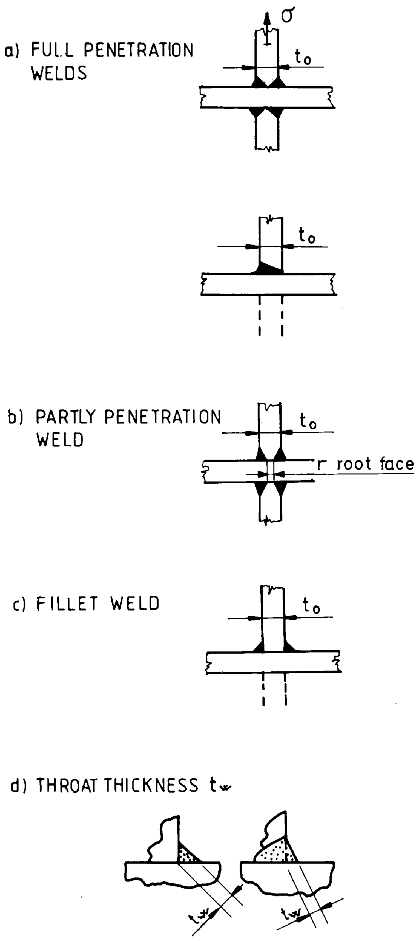

Figure 1
Tee and cross joints

203 The type of connection should be adopted as follows:

a) *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 external welds in way of opening to open sea e.g. pipes, seachests or tee-joints as applicable.

b) *Partly penetration weld*
Connections where the static stress level is high. Acceptable also for dynamically stressed connections, provided the equivalent stress is acceptable, see C300.

c) *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.

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

- oiltight 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.

205 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. With reference to Fig. 2, the various types of intermittent welds are as follows:

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

206 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.

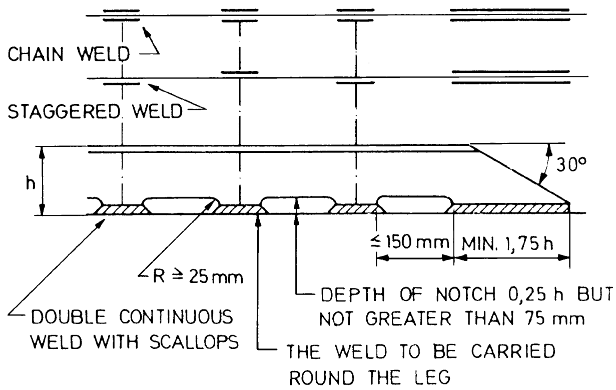

Figure 2
Intermittent welds

B 300 Slot welds

301 Slot weld, see Fig. 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.

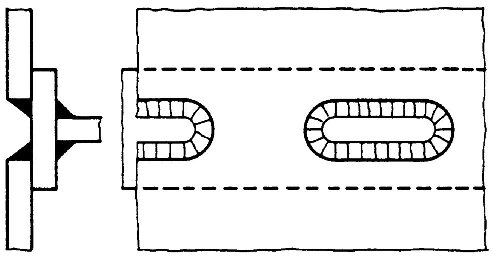

Figure 3
Slot welds

B 400 Lap joint

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

Figure 4
Lap joint

C. Weld size

C 100 General

101 The material factors γ_{Mw} for welded connections are given in Table C1.

Table C1 Material factors γ_{Mw} for welded connections	
Limit states	*Material factor*
ULS	1.3
ALS	1.0

102 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 104.

The yield stress of the weld deposit is in no case to be less than given in DNV-OS-C401.

103 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 C2. If welding consumables with deposits of lower yield stress than specified in Table C2 are used, the applied yield strength shall be clearly informed on drawings and in design reports.

104 The size of some weld connections may be reduced:

— corresponding to the strength of the weld metal, f_w:

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

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

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

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 C2. When deep penetrating welding processes are applied, the required throat thicknesses may be reduced by 15 % provided that sufficient weld penetration is demonstrated.

Table C2 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 $f_r = \left(\frac{f_y}{\sigma_{fw}}\right) \geq 0,75$
Normal strength steels	NV NS	355	1.36	0.75
High strength steels	NV 27NV 32NV 36NV 40	375375375390	1.421.421.421.46	0.750.880.961.00

C 200 Fillet welds

201 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 202 to 204 should be adopted.

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

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

f_r = strength ratio as defined in 104

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

203 The throat thickness of intermittent welds may be as required in 202 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

204 For intermittent welds, the throat thickness is not to exceed:

— for chain welds and scallop welds:

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

t_0 = net thickness abutting plate.

— for staggered welds:

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

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

C 300 Partly penetration welds and fillet welds in cross connections subject to high stresses

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

302 When the abutting plate carries dynamic stresses, the connection shall fulfil the requirements with respect to fatigue, see Sec.6.

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

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

f_w = strength ratio as defined in 104
σ_d = calculated maximum design tensile stress in abutting plate (N/mm^2)
r = root face (mm), see Fig. 1b
t_0 = net thickness (mm) of abutting plate.

minimum 3 mm.

C 400 Connections of stiffeners to girders and bulkheads etc.

401 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.

402 Various standard types of connections between stiffeners and girders are shown in Fig. 5.

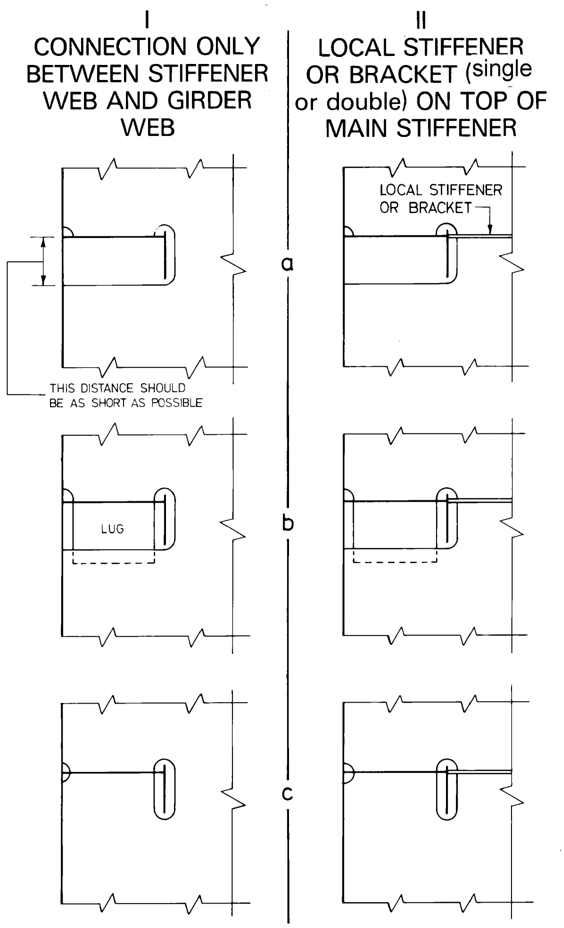

Figure 5
Connections of stiffeners

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

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

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

c = detail shape factor as given in Table C3
f_{yd} = minimum yield design stress (N/mm^2)
l = span of stiffener (m)
s = distance between stiffeners (m)
p_d = design pressure (kN/m^2).

Table C3 Detail shape factor c			
Type ofconnection(see Fig. 5*)*	*IWeb to webconnection only*	*IIStiffener or bracket ontop 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

405 Total weld area a is not to be less than:

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

f_r = strength ratio as defined in 104
a_0 = connection area (mm^2) as given in 404.

The throat thickness is not to exceed the maximum for scallop welds given in 204.

406 The weld connection between stiffener end and bracket is principally to be designed such that the design shear stresses of the connection corresponds to the design resistance.

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

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

C 500 End connections of girders

501 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 300 shall be applied.

502 The end connections of simple girders shall satisfy the requirements for section modulus given for the girder in question.

Where the shear design stresses in web plate exceed 90 N/mm^2, double continuous boundary fillet welds should have throat thickness not less than:

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

τ_d = design shear stress in web plate (N/mm2)
f_w = strength ratio for weld as defined in 104
t_0 = net thickness (mm) of web plate.

C 600 Direct calculation of weld connections

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

602 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.

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

604 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.

605 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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606 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.

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

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

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

$\sigma_{\perp d}$ = 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 Fig. 6
f_u = nominal lowest ultimate tensile strength of the weaker part joined
β_w = appropriate correlation factor, see Table C4
γ_{Mw} = material factor for welds

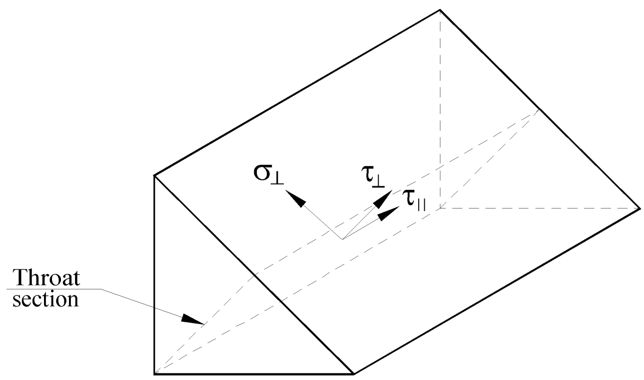

Figure 6
Stresses in fillet weld

Table C4 The correlation factor β_w

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

SECTION 10
CORROSION PROTECTION

A. General

101 In this section the requirements regarding corrosion protection arrangement and equipment are given.

B. Acceptable Corrosion Protection

B 100 Atmospheric zone

101 Steel surfaces in the atmospheric zone shall be protected by coating.

B 200 Splash zone

201 Steel surfaces in the splash zone shall be protected by coating.

202 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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203 For floating units with constant draught, the extent of the splash zone shall extend 5 m above and 4 m below this draught.

204 For bottom fixed structures, such as jackets and TLPs, the definitions given in 205 to 207 apply.

205 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}.

206 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 205
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.

207 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 205
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.

B 300 Submerged zone

301 Steel surfaces in the submerged zone, including splash zone areas below normal operating draught, shall be cathodically protected, preferably in combination with coating. For coated submerged steel the cathodic protection current density can be reduced.

B 400 Internal zone

401 Tanks which are exposed to sea water or other corrosive liquids, typically water ballast tanks, shall be protected by coating. Sacrificial anodes shall be used in combination with coating where relevant, e.g. for water ballast tanks that will stay empty less than approximately 50 % of the time.

402 Tanks which are empty or only partly filled with sea water shall be protected either by coating, corrosion addition or a combination of these methods. De-humidifying equipment can be used for corrosion prevention in spaces designed to be dry.

403 Fresh water tanks shall be coated. Health authorities requirements for certification of the coating with respect to toxicity, taste and smell shall be complied with.

404 Areas with high fatigue utilisation shall be protected by cathodic protection or coating unless the effect of unprotected steel has been accounted for in the fatigue evaluation. Regarding the use of aluminium coating, see D102. To facilitate in-service inspections in ballast tanks and in areas where crack detection is important, light coloured, hard coatings e.g. on epoxy or similar basis shall be used.

405 Magnesium anodes and impressed current systems shall not be used in tanks.

406 Corrosion protection of closed spaces impossible to inspect after final welding is subject to special consideration.

407 Internal surfaces of structural members that may not stay dry or will not be sealed off from the atmosphere, shall be protected by coating. For internal surfaces of compartments that will remain sealed off and dry for the design life of the structure, coating is not required.

408 In ballast tanks which may become gas hazardous areas due to being located adjacent to e.g. fuel tanks or oil storage tanks for liquids with flash point less than 60°C, aluminium anodes shall be so located that the kinetic energy developed in event of their loosening and falling down is less than 275 J. Fillet welds for attachment of anodes shall be continuous and of adequate cross section. Attachment by clamps fixed by set-screws shall not be applied in potentially gas hazardous areas. Attachments by properly secured through-bolts may, however, be applied.

409 Tanks in which anodes are installed shall have sufficient holes for the circulation of air to prevent gas from accumulating in pockets.

B 500 Corrosion additions

501 Unprotected steel (plates, stiffeners and girders) in tanks shall be given a corrosion addition t_k as follows:

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

Guidance note:
Corrosion addition should be used in the splash zone if an ordinary paint coating only is applied to the structure. If a thermally sprayed aluminium coating, glass flake reinforced polyester or

epoxy coating, or similar thick film coating is used, corrosion addition should not be needed.

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502 When a corrosion addition has been added to the scantlings, this shall be clearly noted in design documentation and structural drawings.

C. Cathodic Protection

C 100 General

101 The cathodic protection system shall deliver sufficient protective current to maintain the potential at all steel surfaces of the structure in the submerged zone between – 0.80 V and – 1.10 V versus the Ag/AgCl/sea water reference electrode throughout the design life of the installation.

102 These potentials apply to normal sea water (salinity 32 to 38 g/l) and saline mud.

103 If potential measurements are carried out in brackish water, either a Ag/AgCl reference electrode with closed electrolyte compartment or permeable membrane (not dependent on chloride concentration) or of other type ($Cu/CuSO_4$, Zn or saturated calomel electrode SCE) shall be used.

Guidance note:

The following relationships should be valid between potentials (volts V) measured with Ag/AgCl and other reference cells:

Ag/AgCl/sea water	*Zn*	*$Cu/CuSO_4$*	*SCE*
– 0.80	+ 0.25	– 0.85	– 0.79
– 0.90	+ 0.15	– 0.95	– 0.89
– 1.10	– 0.05	– 1.15	– 1.09

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104 The current density needed to achieve the above potentials shall be selected on the basis of the worst environmental conditions that the unit can be expected to be exposed to with respect to corrosivity.

105 If the unit is operating in an environment with worse conditions than originally intended, additional protection may be required.

106 The reduction in current density for coated surfaces compared with bare steel, will be dependent on the quality of the coating system.

107 Cathodic protection systems for steels with specified minimum yield stress > 550 N/mm^2 are subject to special consideration for applications where hydrogen induced stress cracking (HISC) may be anticipated. See 108 and 109 and Sec.4 D300.

108 Qualification testing shall be carried out for critical applications such as legs and spud cans. Test conditions: Anaerobic, hydrogen sulphide containing environment and cathodic protection potentials of – 1.1 V Ag/AgCl, or more negative potentials. The test procedure, slow strain rate method or similar, should be agreed.

109 If not documented by testing that cathodic protection to – 1.1 V Ag/AgCl is harmless (for steel with specified minimum yield stress higher than 550 N/mm^2), the cathodic protection potential shall be limited by using special anodes of controlled voltage type (with diodes or similar), or other method.

C 200 Protection by sacrificial anodes

201 A cathodic protection system by sacrificial anodes shall be designed to maintain the required potential during the period between complete re-installation of the anodes. This period shall not be less than 5 years.

202 The anodes shall be located so as to give a uniform current distribution to the steel structure.

Guidance note:

Installation of a permanent cathodic protection monitoring system based on potential readings from fixed reference electrodes may be advantageous.

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203 The anode core shall be designed to support the anode and to maintain the anode shape during the later stages of the anode life.

C 300 Protection by impressed current

301 The impressed current anodes shall be located and shielded in such way as to give a protective current distribution being as uniform as possible.

302 Due to the risks of detrimental overprotection, impressed current anodes shall not be located close to areas with high stresses.

303 For complicated structures such as frameworks of pipes, the impressed current system should be designed to provide 1.25 to 1.5 times the calculated current demand, in order to compensate for inefficient current distribution.

304 Installation of a permanent control and monitoring system is required in order to provide adequate cathodic protection and avoid over-protection.

305 The impressed current system should be arranged so that the risk of damages to anodes, cables and reference electrodes is minimised.

C 400 Cathodic protection monitoring system

401 A monitoring system for impressed current cathodic protection systems shall be provided.

402 The monitoring system shall be based on potential readings from fixed reference electrodes and shall be suitable for measuring and recording the level of protection of representative parts of the submerged structure. Locations of reference electrodes should be selected with special attention to areas where under- or overprotection may be expected.

403 It shall be measured that the monitoring system is functioning satisfactorily. If satisfactory functioning can not be proved, potential measurements by divers or submersibles may be required.

404 For steels with specified minimum yield stress higher than 550 N/mm^2 in anaerobic environment, cathodic protection potentials shall be monitored to ensure compliance with the target range indicated in the Guidance note below. In case the target range is exceeded, inspection with respect to possible HISC shall be carried out.

Guidance note:

The target potential range for steels susceptible to HISC in anaerobic, sulphide containing environment is – 770 to – 30 mV versus the Ag/AgCl reference electrode.

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C 500 Testing of effectiveness of corrosion protection system

501 After the cathodic protection system is put into operation, an initial survey shall be performed to establish that all submerged areas are adequately protected. During this survey the structure shall be in normal operating condition. This initial survey shall be carried out within:

— 6 months after delivery for sacrificial anode systems
— 3 months after delivery for impressed current systems.

Guidance note:

Lowering of reference electrode in a line is usually sufficient. Potential readings utilising divers or submersible may be required in special cases.

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D. Coating

D 100 Specification

101 A coating specification shall include description of:

— steel surface treatment for coating application, including shop-primer
— control of temperatures and climatic conditions during blast cleaning and coating application
— coating systems, including coating types, number of coats and film thicknesses
— coating allocation schedule (which coatings where) quality control or inspection requirements.

102 The use of aluminium coating is generally not recommended in tanks for liquids with flash point below 60 °C, in adjacent ballast tanks, in cofferdams, in pump rooms or on decks above the mentioned spaces nor in any other area where gas may accumulate. Organic coatings, e.g. on epoxy basis, containing up to 10 % aluminium by weight in the dry film are, however, acceptable in the mentioned areas.

D 200 Coating application

201 Regarding coating application, see DNV-OS-C401.

SECTION 11
FOUNDATION DESIGN

A. General

A 100 Introduction

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

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

103 Design of foundations shall be based on site specific information, see 200.

104 The partial coefficient method is the selected method in these standards for foundation design (see Sec.2). The application of this method is documented in this section. Alternative methods or safety checking together with general design principles are given in Sec.2.

105 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.

106 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 and structure interaction.

Requirements for the design of the foundation structure itself are given in Sec.4 to Sec.9, as relevant for a foundation structure of steel.

107 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.

108 The definition of limit state categories as given in Sec.2 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 are in this case to 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.

109 The load coefficients to be used for design related to the different groups of limit states are given in Sec.2. Load coefficients for anchor foundations are given in E200.

110 Material coefficients to be used are specified in B to E. The characteristic strength of the soil shall be assessed in accordance with 300.

111 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 C106.

For anchor foundations, material coefficients shall be applied to the characteristic anchor resistance, as described for the respective types of anchors in E.

112 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.

113 Further elaboration on design principles and examples of design solutions for foundation design are given in Classification Note 30.4.

A 200 Site investigations

201 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.

202 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.

203 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.

204 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.

205 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.

206 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.

207 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.

A 300 Characteristic properties of soil

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

302 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.

303 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.

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

305 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.

306 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.

307 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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A 400 Effects of cyclic loading

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

402 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.

403 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 D300.

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

405 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 500.

A 500 Soil and Structure interaction

501 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.

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

503 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 and structure interaction analysis. See Sec.3 I100.

B. Stability of Seabed

B 100 Slope stability

101 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.

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

103 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.

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

B 200 Hydraulic stability

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

202 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.

203 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 300.

B 300 Scour and scour protection

301 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.

302 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.

303 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.

C. Design of Pile Foundations

C 100 General

101 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.

102 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.

103 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.

104 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.

105 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.3 C100 for selection of representative loads.

106 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. 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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107 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 106 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.

108 For calculation of design lateral resistance according to 300, 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.

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

110 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 are normally to be used. Choice of material coefficients is, in such cases, to be in accordance with the determination of characteristic values of shear strength or soil resistance.

C 200 Soil resistance against axial pile loads

201 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.

202 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.

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

204 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.

205 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.

206 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.

207 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.

208 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.

C 300 Soil resistance against lateral pile loads

301 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 108.

302 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.

C 400 Group effects

401 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.

402 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.

D. Design of Gravity Foundations

D 100 General

101 Failure modes within the categories of limit states ULS and ALS shall be considered as described in 200.

102 Failure modes within the SLS, i.e. settlements and displacements, shall be considered as described in 200 using material coefficient γ_M = 1.0.

D 200 Stability of foundations

201 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.

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

203 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.

204 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.

205 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.

206 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.

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

208 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.

209 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.

210 Effects of cyclic loads shall be included by applying load coefficients in accordance with A108.

211 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.

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

D 300 Settlements and displacements

301 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.

302 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.

303 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.

D 400 Soil reaction on foundation structure

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

402 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.

403 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.

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

D 500 Soil modelling for dynamic analysis

501 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.

502 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.

D 600 Filling of voids

601 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.

602 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.

603 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.

E. Design of Anchor Foundations

E 100 General

101 Subsection E applies to the following types of anchor foundations:

— pile anchors
— gravity anchors or suction anchors
— fluke anchors
— drag-in plate anchors
— other types of plate anchors.

102 The analysis of anchor resistance shall be carried out for the ULS and the ALS, in accordance with the safety requirements given in 200. Due consideration shall be given to the specific aspects of the different anchor types and the current state of knowledge and development.

103 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.

104 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.

E 200 Safety requirements for anchor foundations

201 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 must 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.

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

1) 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.
2) Failure may well lead to unacceptable consequences of these types.

203 Load coefficients for the two alternative methods to calculate line tension are given in Table E1 and Table E2 for ULS and ALS, respectively. For mooring in deep water a dynamic analysis is required.

Table E1 Load coefficients for ULS

Consequence class	*Type of analysis*	γ_{mean}	γ_{dyn}
1	Dynamic	1.10	1.50
2	Dynamic	1.40	2.10
1	Quasi-static	1.70	
2	Quasi-static	2.50	

Table E2 Load coefficients for ALS

Consequence class	*Type of analysis*	γ_{mean}	γ_{dyn}
1	Dynamic	1.00	1.10
2	Dynamic	1.00	1.25
1	Quasi-static	1.10	
2	Quasi-static	1.35	

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

$$T_d = T_{C-mean} \cdot \gamma_{mean} + T_{C-dyn} \cdot \gamma_{dyn}$$

$T_{C\text{-mean}}$ = the characteristic mean line tension due to pretension (T_{pre}) and the effect of mean environmental loads in the environmental state

$T_{C\text{-dyn}}$ = the characteristic dynamic line tension equal to the increase in tension due to oscillatory low-frequency and wave-frequency effects.

205 Material coefficients for use in combination with the load coefficients in Table E1 and Table E2 are given specifically for the respective types of anchors in 300 to 600.

E 300 Pile anchors, gravity and suction anchors

301 Pile anchors shall be designed in accordance with the relevant requirements given in C.

302 Gravity anchors shall be designed in accordance with the relevant requirements given in D. 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.

303 Suction anchors are vertical cylindrical anchors with closed top, which are penetrated mainly by application of underpressure (suction) in that closed compartment. They are in principle to be designed as gravity anchors with skirts, although their length/diameter ratio and the position of the anchor padeye may introduce different failure modes. The characteristic anchor resistance shall be conservatively assessed accounting for the quality of the soil investigation and the complexity of the soil conditions.

304 The material coefficients to be applied to the resistance of pile anchors, gravity anchors and suction anchors shall not be taken less than 1.3 for the ULS and 1.0 for the ALS.

E 400 Fluke anchors

401 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.

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

403 Fluke anchors are normally to 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 DNV-RP-E301.

404 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 DNV-RP-E301. For fluke anchors in sand the same load coefficients as given in 200 should be applied, and the target installation load should normally not be taken less than the design load.

405 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 material coefficient γ_M on this predicted component of the anchor resistance shall then be 1.3 for the ULS and 1.0 for the ALS. See DNV-RP-E301 for details.

E 500 Drag-in plate anchors

501 Drag-in plate anchors have been developed for use in combination with taut mooring systems and they can resist both the vertical and the horizontal loads transferred to the anchors in such a system.

502 This anchor is installed as a conventional fluke anchor and when the target installation tension T_i has been reached it is triggered to create normal loading against the fluke (plate). In this normal loading mode the anchor acts as an embedded plate with a high pullout resistance R_p. The performance ratio, $P_r = R_p/T_i$, is a significant design parameter, see the recommended design procedure in DNV-RP-E302.

503 According to this design procedure the anchor pullout resistance R_p is split into a static component R_S and a cyclic component ΔR_{cy}. The design anchor resistance is obtained by multiplying the characteristic value of the respective component by a material coefficient, $\gamma_{M,1}$ on the static component RS and $\gamma_{M,2}$ on the dynamic component ΔR_{cy} as explained in more detail in DNV-RP-E302.

504 According to DNV-RP-E302 the material coefficients are:

a) *For the ULS:*Dynamic analysis: $\gamma_{M,1}$ = 1.15 and $\gamma_{M,2}$ = 1.40Quasi-static analysis: $\gamma_{M,1} = \gamma_{M,2}$ = 1.15.
b) *For the ALS:*Dynamic analysis: $\gamma_{M,1}$ = 1.00 and $\gamma_{M,2}$ = 1.15Quasi-static analysis: $\gamma_{M,1} = \gamma_{M,2}$ = 1.00.

The values given above for $\gamma_{M,1}$ assume that the target installation tension T_i has been measured satisfactorily during anchor installation, e.g. by the DNV Tentune method, see DNV-RP-E302.

E 600 Other types of plate anchors

601 Design methodologies for other types of plate anchors like 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. However, the design procedure described in DNV-RP-E302 for drag-in plate anchors may be used (with caution) as a guidance for assessment of the pullout resistance also of other types of plate anchors.

602 Tentatively, the same material coefficients as given for drag-in plate anchors in 504 apply also for other types of plate anchors.

APPENDIX A
CROSS SECTIONAL TYPES

A. Cross Sectional Types

A 100 General

101 Cross sections of beams are divided into different types dependent of their ability to develop plastic hinges as given in Table A1.

Table A1 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

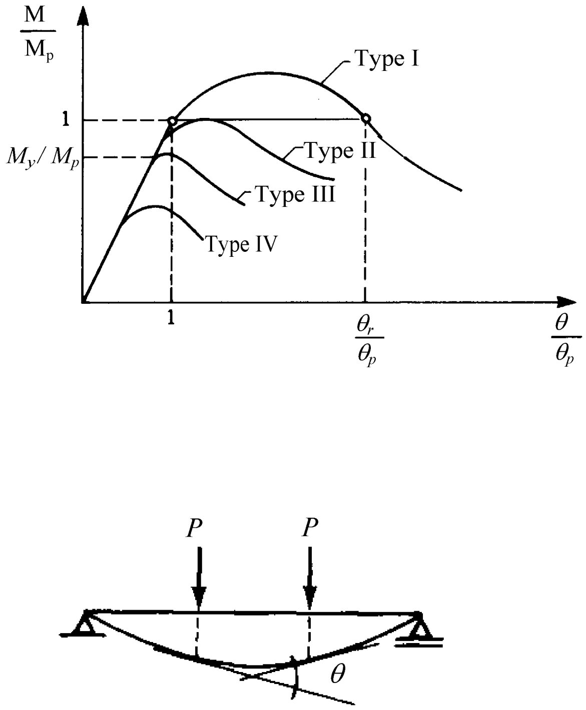


Figure 1
Relation between moment M and plastic moment resistance M_p, and rotation θ for cross sectional types. M_y is elastic moment resistance

102 The categorisation of cross sections depends on the proportions of each of its compression elements, see Table A3.

103 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.

104 The various compression elements in a cross section such as web or flange, can be in different classes.

105 The selection of cross sectional type is normally quoted by the highest or less favourable type of its compression elements.

A 200 Cross section requirements for plastic analysis

201 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.

202 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.

A 300 Cross section requirements when elastic global analysis is used

301 When elastic global analysis is used, the role of cross section classification is to identify the extent to which the resistance of a cross section is limited by its local buckling resistance.

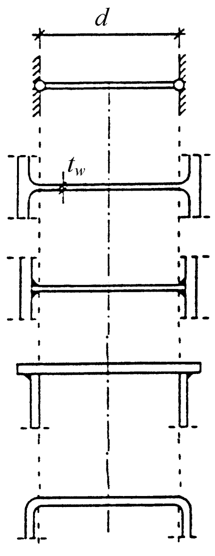
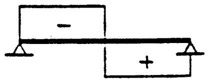
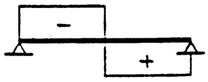
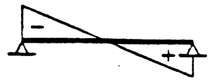
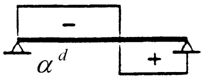
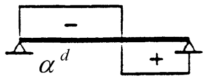
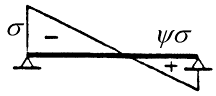
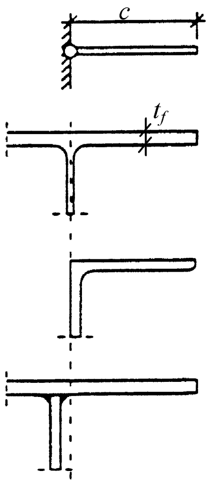
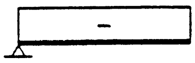
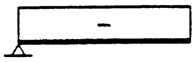
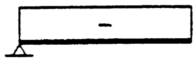
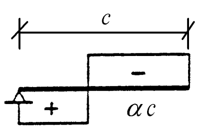
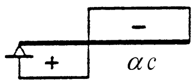
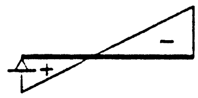
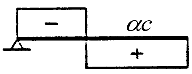
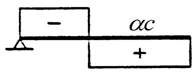
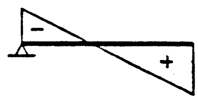
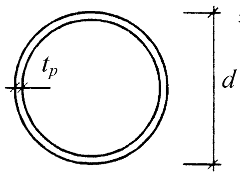
302 When all the compression elements of a cross section are type III, its resistance may be based on an elastic distribution of stresses across the cross section, limited to the yield strength at the extreme fibres.

Table A2 Coefficient related to relative strain	
NV Steel grade [1)]	ε [2)]
NV-NS	1
NV-27	0.94
NV-32	0.86
NV-36	0.81
NV-40	0.78
NV-420	0.75
NV-460	0.72
NV-500	0.69
NV-550	0.65
NV-620	0.62
NV-690	0.58

1) The table is not valid for steel with improved weldability. See Sec.4, Table D1, footnote 1).

2) $\varepsilon = \sqrt{\frac{235}{f_y}}$ where f_y is yield strength

Table A3 Maximum width to thickness ratios for compression elements

Cross section part	*Type I*	*Type II*	*Type III*
	 $d / t_w \le 33\,\varepsilon$ [1)]	 $d / t_w \le 38\,\varepsilon$	 $d / t_w \le 42\,\varepsilon$
	 $d / t_w \le 72\,\varepsilon$ [2)]	 $d / t_w \le 83\,\varepsilon$	 $d / t_w \le 124\,\varepsilon$
D = h - 3 t_w [3)]	 when $\alpha > 0.5$: $d/t_w \le \dfrac{396\varepsilon}{13\alpha-1}$ when $\alpha \le 0.5$: $d/t_w \le \dfrac{36\varepsilon}{\alpha}$	 when $\alpha > 0.5$: $d/t_w \le \dfrac{456\varepsilon}{13\alpha-1}$ when $\alpha \le 0.5$: $d/t_w \le \dfrac{41.5\varepsilon}{\alpha}$	 when $\psi > -1$: $d/t_w \le \dfrac{126\varepsilon}{2+\psi}$ when $\psi \le -1$: $d/t_w \le 62\varepsilon(1-\psi)\sqrt{\lvert\psi\rvert}$
	 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: $(c/t_f) \le 14\varepsilon$
	 Rolled: $c/t_f \le 10\varepsilon/\alpha$ Welded: $c/t_f \le 9\varepsilon/\alpha$	 Rolled: $(c/t_f) \le 10\varepsilon/\alpha$ Welded: $c/t_f \le 9\varepsilon/\alpha$	 Rolled: $(c/t_f) \le 23\varepsilon\sqrt{C}$ [4)] Welded: $c/t_f \le 21\varepsilon\sqrt{C}$
	 Rolled: $(c/t_f) \le \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$ Welded: $c/t_f \le \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$	 Rolled: $(c/t_f) \le \dfrac{11\varepsilon}{\alpha\sqrt{\alpha}}$ Welded: $(c/t_f) \le \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$	 Rolled: $(c/t_f) \le 23\varepsilon\sqrt{C}$ Welded: $c/t_f \le 21\varepsilon\sqrt{C}$
	$d / t_p \le 50\,\varepsilon^2$	$d / t_p \le 70\,\varepsilon^2$	$d / t_p \le 90\,\varepsilon^2$

1) Compression negative
2) ε is defined in Table A2
3) Valid for rectangular hollow sections (RHS)
4) C is the buckling coefficient. See e.g. Classification Note 30.1, Table 3.2, No. 4 and 7 or Eurocode 3 Table 5.3.3 (denoted k_σ)
5) Valid for axial and bending, not external pressure.