OFFSHORE STANDARD
DNV-OS-C101

# DESIGN OF OFFSHORE STEEL STRUCTURES, GENERAL (LRFD METHOD)

APRIL 2011

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.

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— *Standards.* Technical requirements.
— *Recommended Practices.* Guidance.

The Standards and Recommended Practices are offered within the following areas:

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B) Materials Technology
C) Structures
D) Systems
E) Special Facilities
F) Pipelines and Risers
G) Asset Operation
H) Marine Operations
J) Cleaner Energy
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# CHANGES

- **General**

This document supersedes DNV-OS-C101, October 2008.
This update is based on experience and feedback gained during 10 years since first edition of document.

## Main changes in April 2011

- **General**

— A number of editorial corrections and clarifications have been made.

- **Sec.4 Structural Categorisation, Material Selection and Inspection Principles**

— In sub-section element D500 a new paragraph has been added concerning PWHT.

- **Sec.9 Weld Connections**

— Item B203 has been amended, mainly list-item a) concerning full penetration weld.

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# SECTION 1
# INTRODUCTION

## A. General

### A 100 Introduction

**101** This offshore standard provides principles, technical requirements and guidance for the structural 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 may also be used as a stand-alone document for objects where no object standard exist.

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

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

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

### A 200 Objectives

**201** 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.

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

### A 400 Other than 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 latest revision of the documents shall be applied, unless otherwise specified.

**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.

### A 500 Classification

**501** Classification principles, procedures and applicable class notations related to classification services of

offshore units are specified in the DNV Offshore Service Specifications given in Table A1.

| **Table A1 DNV Offshore Service Specifications** | |
|---|---|
| *Reference* | *Title* |
| DNV-OSS-101 | Rules for Classification of Offshore Drilling and Support Units |
| DNV-OSS-102 | Rules for Classification of Floating Production, Storage and Loading Units |
| DNV-OSS-103 | Rules for Classification of LNG/LPG Floating Production and Storage Units or Installations |
| DNV-OSS-121 | Classification Based on Performance Criteria Determined by Risk Assessment Methodology |

**502** Documentation for classification shall be in accordance with the NPS DocReq (DNV Nauticus Production System for documentation requirements) and DNV-RP-A201.

**503** In case of classification, the Limit States ULS, FLS, and ALS apply. Technical requirements given in Sec.8, related to Serviceability Limit States (SLS), are not required to be fulfilled as part of classification.

## B. References

### B 100 General

**101** The DNV documents in Tables B1 and B2 and recognised codes and standards in Table B3 are referred to in this standard.

**102** The latest valid revision of the DNV reference documents in Tables B1 and B2 applies. These include acceptable methods for fulfilling the requirements in this standard. See also current DNV List of Publications.

**103** When designing a unit where an object standard exists, the object standard for the specific type of unit shall be applied, see Table B2. The object standard gives references to this standard when appropriate, see also A103 and A104.

**104** Other recognised codes or standards may be applied provided it is shown that they meet or exceed the level of safety of the actual DNV Offshore Standard.

| **Table B1 DNV Reference Documents** | |
|---|---|
| *Reference* | *Title* |
| DNV-OS-A101 | Safety Principles and Arrangement |
| DNV-OS-B101 | Metallic Materials |
| DNV-OS-C401 | Fabrication and Testing of Offshore Structures |
| DNV-OS-E301 | Position Mooring |
| | Rules for Planning and Execution of Marine Operations |
| DNV-RP-A201 | Plan Approval Documentation Types – Definitions |
| DNV-RP-B401 | Cathodic Protection Design |
| DNV-RP-C201 | Buckling Strength of Plated Structures |
| DNV-RP-C202 | Buckling Strength of Shells |
| DNV-RP-C203 | Fatigue Strength Analysis of Offshore Steel Structures |
| DNV-RP-C204 | Design against Accidental Loads |
| DNV-RP-C205 | Environmental Conditions and Environmental Loads |
| DNV-RP-E301 | Design and Installation of Fluke Anchors in Clay |
| DNV-RP-E302 | Design and Installation of Plate Anchors in Clay |
| DNV-RP-E303 | Geotechnical Design and Installation of Suction Anchors in Clay |
| Classification Note 30.1 | Buckling Strength Analysis of Bars and Frames, and Spherical Shells, Sec.2 Bars and Frames |
| Classification Note 30.4 | Foundations |
| Classification Note 30.6 | Structural Reliability Analysis of Marine Structures |
| Classification Note 30.7 | Fatigue Assessments of Ship Structures |

| Table B2 DNV Offshore Object 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 |
| DNV-OS-C107 | Structural Design of Ship-shaped Drilling and Well Service Units |

| Table B3 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 |
| 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 |

## C. Definitions

### C 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.

### C 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 surfaces of the unit above the splash zone.

**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 the applicable 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 allowance:* Extra wall thickness added during design to compensate for any anticipated reduction in thickness during the operation.

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

**212** *Design temperature:* The design temperature for a unit is the reference temperature for assessing areas where the unit can be transported, installed and operated. The design temperature is to be lower or equal to the *lowest mean daily temperature* in air for the relevant areas. For seasonal restricted operations the *lowest mean daily temperature* in air for the season may be applied.

**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** *Foundation:* A device transferring loads from a heavy or loaded object to the vessel structure.

**221** *Guidance note:* Information in the standard added in order to increase the understanding of the requirements.

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

**223** *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.

**224** *Limit State:* A state beyond which the structure no longer satisfies the requirements. The following categories of limit states are of relevance for structures:

— ULS = ultimate limit states
— FLS = fatigue limit states
— ALS = accidental limit states
— SLS = serviceability limit states.

**225** *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.

**226** *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.

**227** *Lowest mean daily temperature:* The lowest value on the annual mean daily average temperature curve for the area in question. For temporary phases or restricted operations, the lowest mean daily temperature may be defined for specific seasons.

*Mean daily average temperature:* the statistical mean average temperature for a specific calendar day.

*Mean:* statistical mean based on number of years of observations.

*Average:* average during one day and night.

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

**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 B2.

**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 production, drilling or other similar operations, and combined environmental and operational loadings are within the appropriate design limits established for such operations (including normal operations, survival, accidental).

**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 may 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** *Service temperature:* Service temperature is a reference temperature on various structural parts of the unit used as a criterion for the selection of steel grades.

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

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

**243** *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.

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

**245** *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.

**246** *Splash zone:* The external surfaces of the unit that are periodically in and out of the water. The determination of the splash zone includes evaluation of all relevant effects including influence of waves, tidal variations, settlements, subsidence and vertical motions, see Sec.10 B200.

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

**248** *Supporting structure:* Strengthening of the vessel structure, e.g. a deck, in order to accommodate loads and moments from a heavy or loaded object.

**249** *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.

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

**251** *Temporary conditions:* Design conditions not covered by *operating conditions*, e.g. conditions during fabrication, mating and installation phases, transit phases, accidental.

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

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

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

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

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

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

## D. Abbreviations and Symbols

### D 100 Abbreviations

**101** Abbreviations as shown in Table D1 are used in this standard.

| **Table D1 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 |
| DFF | design fatigue factor |
| DNV | Det Norske Veritas |
| EHS | extra high strength |
| FLS | fatigue limit state |
| FM | Fracture mechanics |
| HAT | highest astronomical tide |
| HISC | hydrogen induced stress cracking |
| HS | high strength |
| ISO | international organisation of standardisation |
| LAT | lowest astronomic tide |
| LRFD | load and resistance factor design |
| MPI | magnetic particle inspection |
| MSL | mean sea level |
| NACE | National Association of Corrosion Engineers |
| NDT | non-destructive testing |
| NS | normal strength |
| PWHT | post weld heat treatment |
| RP | recommended practise |
| RHS | rectangular hollow section |
| SCE | saturated calomel electrode |
| SCF | stress concentration factor |
| SLS | serviceability limit state |
| SMYS | specified minimum yield stress |
| SRB | sulphate reducing bacteria |
| TLP | tension leg platform |
| ULS | ultimate limit states |
| WSD | working stress design |

### D 200 Symbols

**201** *Latin characters*

$\alpha_0$ connection area
$a_v$ vertical acceleration
b full breadth of plate flange
$b_e$ effective plate flange width
c detail shape factor
d bolt diameter
f load distribution factor
$f_E$ elastic buckling stress
$f_r$ strength ratio
$f_u$ nominal lowest ultimate tensile strength
$f_{ub}$ ultimate tensile strength of bolt

$f_w$ strength ratio
$f_y$ specified minimum yield stress
$g$, $g_o$ acceleration due to gravity
$h$ height
$h_{op}$ vertical distance from the load point to the position of maximum filling height
$k_a$ correction factor for aspect ratio of plate field
$k_m$ bending moment factor
$k_{pp}$ fixation parameter for plate
$k_{ps}$ fixation parameter for stiffeners
$k_s$ hole clearance factor
$k_t$ shear force factor
$l$ stiffener span
$l_o$ distance between points of zero bending moments
$n$ number
$p$ pressure
$p_d$ design pressure
$r$ root face
$r_c$ radius of curvature
$s$ distance between stiffeners
$t_0$ net thickness of plate
$t_k$ corrosion addition
$t_w$ throat thickness
$A_s$ net area in the threaded part of the bolt
$C$ weld factor
$C_e$ factor for effective plate flange
$D$ deformation load
$E$ environmental load
$F_d$ design load
$F_k$ characteristic load
$F_{pd}$ design 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*

| | |
|---|---|
| $\alpha$ | angle between the stiffener web plane and the plane perpendicular to the plating |
| $\beta_w$ | correlation factor |
| $\delta$ | deflection |
| $\phi$ | resistance factor |
| $\gamma_f$ | load factor |
| $\gamma_M$ | material factor (material coefficient) |
| $\gamma_{Mw}$ | material factor for welds |
| $\lambda$ | reduced slenderness |
| $\theta$ | rotation angle |
| $\mu$ | friction coefficient |
| $\rho$ | density |
| $\sigma_d$ | design stress |
| $\sigma_{fw}$ | characteristic yield stress of weld deposit |
| $\sigma_{jd}$ | equivalent design stress for global in-plane membrane stress |
| $\sigma_{pd1}$ | design bending stress |
| $\sigma_{pd2}$ | design bending stress |
| $\tau_d$ | 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** The overall structural safety shall be evaluated on the basis of preventive measures against structural failure put into design, fabrication and in-service inspection as well as the unit's residual strength against total collapse in the case of structural failure of vital elements.

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

**105** 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 for welded structures, ref. DNV-RP-C203.

**106** 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.

**107** 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 states (ULS)* corresponding to the ultimate resistance for carrying loads.
— *Fatigue limit states (FLS)* related to the possibility of failure due to the effect of cyclic loading.
— *Accidental limit states (ALS)* corresponding to damage to components due to an accidental event or operational failure.
— *Serviceability limit states (SLS)* corresponding to the criteria applicable to normal use or durability.

**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 safety level is obtained as closely as possible by applying load and resistance factors to characteristic reference values of the basic variables. The basic variables are, in this context, defined as:

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

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

### 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
$\gamma_f$ = 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:

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

$S_{ki}$ = 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
$\phi$ = resistance factor.

The resistance factor relate to the material factor $\gamma_M$ as follows:

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

$\gamma_M$ = 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 5$^{th}$ 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 operating 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 $\gamma_f$ for ULS**

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

Load categories are:
G = permanent load
Q = variable functional load
E = environmental load
D = deformation load
For description of load categories see Sec.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 $\gamma_f$ = 1.0 on G and Q loads in combination a) results in higher design load effect, the load factor of 1.0 shall be used.

**404** Based on a safety assessment considering the risk for both human life and the environment, the load factor $\gamma_f$ 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 $\gamma_f$ in the FLS is 1.0 for all load categories.

### D 600 Load factor for SLS

**601** For analyses of SLS the load factor $\gamma_f$ is 1.0 for all load categories, both for temporary and operating design conditions.

### D 700 Load factor for ALS

**701** The load factors $\gamma_f$ 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.

**103** For structural arrangement of mooring equipment and arrangement/devices for towing, see DNV-OS-E301 Ch.2 Sec.4 N-O. The mooring and towing equipment, including the support to main structure, shall be designed for the loads and acceptance criteria specified in DNV-OS-E301 Ch.2 Sec.4.

## 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 operating 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.<br>See DNV Rules for Planning and Execution of Marine Operations. | | | | | |

**Table B2 Basis for selection of characteristic loads for operating design conditions**

| *Load category* | *Limit states – operating design conditions* | | | | |
|---|---|---|---|---|---|
| | *ULS* | *FLS* | *ALS* | | *SLS* |
| | | | *Intact structure* | *Damaged structure* | |
| Permanent (G) | Expected value | | | | |
| Variable (Q) | Specified value | | | | |
| Environmental (E) | Annual probability[1)] being exceeded = $10^{-2}$ (100 year return period) | Expected load history | Not applicable | Load with return period not less than 1 year | Specified value |
| Accidental (A) | | | Specified value, see also DNV-OS-A101 | | |
| Deformation (D) | Expected extreme value | | | | |
| 1) The joint probability of exceedance applies, see F. | | | | | |

## 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 load p (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, crew spaces | 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 structure shall be designed to resist the maximum hydrostatic pressure of the heaviest filling in tanks that may occur during fabrication, installation and operation.

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

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

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

**305** Pumping pressures may be limited by installing appropriate alarms and auto-pump cut-off system (i.e. high level and high-high level with automatic stop of the pumps). In such a situation the pressure head may be taken to be the cut-off pressure head.

Descriptions and requirements related to different tank arrangements are given in DNV-OS-D101 Ch.2 Sec.3 C300.

**306** Dynamic pressure heads due to flow through pipes shall be considered, see 308.

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

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

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

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

$\rho$ = density of liquid (t/m$^3$)

$g_0$ = 9.81 m/s$^2$

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

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

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

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

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

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

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

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

### D 400 Lifeboat platforms

**401** Lifeboat platforms shall be checked for the ULS and ALS condition if relevant. A dynamic factor of $0.2 \cdot g_0$ due to retardation of the lifeboats when lowered shall be included in both ULS and ALS condition.

## 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 DNV-RP-C205.

### 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 defined in the design basis or design brief, and stated in the unit's Operation Manual.

**202** The North Atlantic scatter diagram should be used in ULS, ALS 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.

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

### 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 should be determined according to DNV-RP-C205.

**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 chords and bracings, tendons, risers) where the Morison equation is applicable, the wave loads should be estimated by selection of drag and inertia coefficients as specified in DNV-RP-C205.

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

### 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 DNV-RP-C205.

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---

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

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

### E 800 Vortex induced oscillations

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

### E 900 Current

**901** 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 DNV-RP-C205.

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---

### E 1000 Tidal effects

**1001** 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.

**1002** 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.

### E 1100 Marine growth

**1101** 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.

**1102** Effect of marine growth shall be considered, where relevant.

### E 1200 Snow and ice accumulation

**1201** Ice accretion from sea spray, snow, rain and air humidity shall be considered, where relevant.

**1202** Snow and ice loads may be reduced or neglected if a snow and ice removal procedures are established.

**1203** When determining wind and hydrodynamic load, possible increases of cross-sectional area and changes in surface roughness caused by icing shall be considered, where relevant.

**1204** For buoyant structures the possibility of uneven distribution of snow and ice accretion shall be considered.

### E 1300 Direct ice load

**1301** 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.

**1302** 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 1400 Water level, settlements and erosion

**1401** 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.

**1402** Uncertainty of measurements and possible erosion shall be considered.

### E 1500 Earthquake

**1501** Relevant earthquake effects shall be considered for bottom fixed structures.

**1502** 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.

**1503** 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.

**1504** The dynamic characteristics of the structure and its foundation should be determined using a three-dimensional analytical model. A two-dimensional or axis-symmetric model may be used for the soil and structure interaction analysis provided compatibility with the three-dimensional structural model is ensured.

**1505** Where characteristic ground motions, soil characteristics, damping and other modelling parameters are subject to great uncertainties, a parameter sensitivity study should be carried out.

**1506** 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.

## F. Combination of Environmental Loads

### F 100 General

**101** Where applicable data are available joint probability of environmental load components, at the specified probability level, may be considered. Alternatively, joint probability of environmental loads may be approximated by combination of characteristic values for different load types as shown in Table F1.

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

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

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

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

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

**Table 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
— 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 for permanently located bottom founded units

**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:

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

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

**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 various object standards.

# SECTION 4
# STRUCTURAL CATEGORISATION, MATERIAL SELECTION AND INSPECTION PRINCIPLES

## A. General

### A 100 Scope

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

## B. Temperatures for Selection of Material

### B 100 General

**101** The design temperature for a unit is the reference temperature for assessing areas where the unit can be transported, installed and operated.

The design temperature is to be lower or equal to the *lowest mean daily temperature* in air for the relevant areas. For seasonal restricted operations the *lowest mean daily temperature* in air for the season may be applied.

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

**103** The service temperature for various structural parts is given in B200 and B300. In case different service temperatures are defined in B200 and B300 for a structural part the lower specified value shall be applied. Further details regarding service temperature for different structural elements are given in the various object standards.

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

### B 200 Floating units

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

**202** External structures below the lowest waterline need not be designed for service temperatures lower than 0°C.

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

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

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

A higher service temperature may be accepted if adequate supporting data can be presented relative to *lowest mean daily temperature* applicable to the relevant actual 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 suitable fracture toughness for the actual service temperature and thickness is selected. Then a proper fabrication method is used. In special cases post weld heat treatment may be performed to reduce crack driving stresses, see D501 and 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.

**307** The extent of NDT for welds in block joints and erection joints transverse to main stress direction shall not be less than for IC II.

## 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 service 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, see D401 and DNV-OS-C401.

**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** Different steel grades are defined within each strength group, depending upon the required impact toughness properties. The grades are referred to as A, B, D, E, and F for normal weldability, and AW, BW, DW, and EW for improved weldability, 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*[3] *(°C)* |
|---|---|---|---|
| | *Normal weldability* | *Improved weldability*[2] | |
| 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 are subject to agreement between the contracting parties for thickness of 25 mm or less.

2) For steels with improved weldability, through-thickness properties are specified, see DNV-OS-B101.

3) Charpy V-notch impact tests, see DNV-OS-B101.

## D 300 Selection of structural steel

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

**Table D3 Thickness limitations (mm) of structural steels for different structural categories and service temperatures (°C)**

| *Structural Category* | *Grade* | *≥ 10* | *0* | *-10* | *-20* | *-25* | *-30* |
|---|---|---|---|---|---|---|---|
| Secondary | A | 35 | 30 | 25 | 20 | 15 | 10 |
| | B/BW | 70 | 60 | 50 | 40 | 30 | 20 |
| | D/DW | 150 | 150 | 100 | 80 | 70 | 60 |
| | E/EW | 150 | 150 | 150 | 150 | 120 | 100 |
| | AH/AHW | 60 | 50 | 40 | 30 | 20 | 15 |
| | DH/DHW | 120 | 100 | 80 | 60 | 50 | 40 |
| | EH/EHW | 150 | 150 | 150 | 150 | 120 | 100 |
| | FH | 150 | 150 | 150 | 150 | *) | *) |
| | AEH | 70 | 60 | 50 | 40 | 30 | 20 |
| | DEH/DEHW | 150 | 150 | 100 | 80 | 70 | 60 |
| | EEH/EEHW | 150 | 150 | 150 | 150 | 120 | 100 |
| | FEH | 150 | 150 | 150 | 150 | *) | *) |
| Primary | A | 30 | 20 | 10 | N.A. | N.A. | N.A. |
| | B/BW | 40 | 30 | 25 | 20 | 15 | 10 |
| | D/DW | 70 | 60 | 50 | 40 | 35 | 30 |
| | E/EW | 150 | 150 | 100 | 80 | 70 | 60 |
| | AH/AHW | 30 | 25 | 20 | 15 | 12.5 | 10 |
| | DH/DHW | 60 | 50 | 40 | 30 | 25 | 20 |
| | EH/EHW | 120 | 100 | 80 | 60 | 50 | 40 |
| | FH | 150 | 150 | 150 | 150 | *) | *) |
| | AEH | 35 | 30 | 25 | 20 | 17.5 | 15 |
| | DEH/DEHW | 70 | 60 | 50 | 40 | 35 | 30 |
| | EEH/EEHW | 150 | 150 | 100 | 80 | 70 | 60 |
| | FEH | 150 | 150 | 150 | 150 | *) | *) |
| Special | D/DW | 35 | 30 | 25 | 20 | 17.5 | 15 |
| | E/EW | 70 | 60 | 50 | 40 | 35 | 30 |
| | AH/AHW | 15 | 10 | N.A. | N.A. | N.A. | N.A. |
| | DH/DHW | 30 | 25 | 20 | 15 | 12.5 | 10 |
| | EH/EHW | 60 | 50 | 40 | 30 | 25 | 20 |
| | FH | 120 | 100 | 80 | 60 | 50 | 40 |
| | AEH | 20 | 15 | 10 | N.A. | N.A | N.A. |
| | DEH/DEHW | 35 | 30 | 25 | 20 | 17.5 | 15 |
| | EEH/EEHW | 70 | 60 | 50 | 40 | 35 | 30 |
| | FEH | 150 | 150 | 100 | 80 | 70 | 60 |

*) For service temperature below -20°C the upper limit for use of this grade must be specially considered.

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 service temperature above 10 °C may be specially considered.

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

### D 400 Fracture mechanics (FM) testing

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

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

For details on FM testing methods, see DNV-OS-C401 Ch.2 Sec.1 C900.

### D 500 Post weld heat treatment (PWHT)

**501** For units which are intended to operate continuously at the same location for more than 5 years, PWHT shall be applied for joints in C-Mn steels in special areas when the material thickness at the welds exceeds 50mm. For details, see DNV-OS-C401 Ch.2 Sec.2 F200.

If, however, satisfactory performance in the as-welded condition can be documented by a fitness-for-purpose assessment applying fracture mechanics testing, fracture mechanics and fatigue crack growth analyses, PWHT may be omitted.

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

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

**105** Gross scantlings may be utilised in the calculation of hull structural strength, provided a corrosion protection system in accordance with Sec.10 is installed and maintained.

**106** In case corrosion protection in accordance with Sec.10 is not installed (and maintained) corrosion additions as given in Sec.10 B407 shall be used. The corrosion addition shall not be accounted for in the determination of stresses and resistance for local capacity checks.

### 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 are possible modes of failure shall be investigated for yielding.

Individual design stress components and the von Mises equivalent design stress for plated structures shall not exceed the design resistance (Sec.2 D200).

**Guidance note:**

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

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

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

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

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

**Guidance note:**

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

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

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**403** For yield check of welded connections, see Sec.9.

### A 500 Buckling check

**501** Elements of cross sections not fulfilling requirements to cross section type III shall be checked for local buckling.

Cross sectional types are defined in Appendix A.

**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 DNV-RP-C201 for plated structures, DNV-RP-C202 for shells, or Classification Note 30.1 for bars and frames, the tolerance requirements given in DNV-OS-C401 should not be exceeded, unless specifically documented.

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## B. Flat Plated Structures and Stiffened Panels

### B 100 General

**101** The material factor $\gamma_M$ 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 DNV-RP-C201.

### 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.

**Guidance note:**

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

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**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 | $\gamma_{M0}$ | 1.15 |
| Resistance of Class 4 cross sections | $\gamma_{M1}$ | 1.15 |
| Resistance of members to buckling | $\gamma_{M1}$ | 1.15 |

1) Symbols according to Eurocode 3.

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

## C. Shell Structures

### C 100 General

**101** The buckling stability of cylindrical and un-stiffened conical 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 $\gamma_M$ for buckling**

| *Type of structure* | $\lambda \le 0.5$ | $0.5 < \lambda < 1.0$ | $\lambda \ge 1.0$ |
|---|---|---|---|
| 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.

| | | |
|---|---|---|
| $\lambda$ | = | reduced slenderness parameter |
| | = | $\sqrt{\frac{f_y}{f_E}}$ |
| $f_y$ | = | specified minimum yield stress |
| $f_E$ | = | elastic buckling stress for the buckling mode under consideration. |

## D. Tubular Members, Tubular Joints and Conical Transitions

### D 100 General

**101** Tubular members may be checked according to Classification Note 30.1, API RP 2A - LRFD or NORSOK N-004.

For interaction between local shell buckling and column buckling, and effect of external pressure, DNV-RP-C202 may be used.

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

**Guidance note:**

a) Effect of local buckling of tubular members without external pressure (i.e. subject to axial force and/or bending moment) are given in Appendix A, cross section type IV. Section 3.8 of DNV-RP-C202 may be used, see C100.

b) Effect of local buckling of tubular members with external pressure need not be considered for the following

diameter (D) to thickness (t) ratio:

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

where

E = modulus of elasticity, and
$f_y$ = specified minimum yield stress.

In case of local shell buckling, see C100, section 3.8 of DNV-RP-C202, API RP 2A - LRFD or NORSOK N-004 may be used.

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**103** Tubular joints and conical transitions may be checked according to API RP 2A - LRFD or NORSOK N-004.

**104** The material factor $\gamma_M$ 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.

**Guidance note:**

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

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**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:

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

$f_{yd}$ = design yield strength $f_y/\gamma_M$
$f_y$ 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
$\gamma_M$ = 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:

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

$k_a$ = 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 $s/l = 1.0$

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

$p_d$ = design pressure (kN/m$^2$) as given in Sec.3

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

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

$\sigma_{jd}$ = equivalent design stress for global in-plane membrane stress:

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

$k_{pp}$ = fixation parameter for plate
= 1.0 for clamped edges
= 0.5 for simply supported edges.

**Guidance note:**

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

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## F 400 Stiffeners

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

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

$l$ = stiffener span (m)

$k_m$ = bending moment factor, see Table G1

$\sigma_{pd2}$ = design bending stress (N/mm$^2$)
= $f_{yd} - \sigma_{jd}$

$k_{ps}$ = fixation parameter for stiffeners
= 1.0 if at least one end is clamped
= 0.9 if both ends are simply supported.

**Guidance note:**

The section modulus $Z_s$ will typically be calculated for plate side and stiffener flange side.

The design bending stress $\sigma_{pd2}$ for plate side is given as a linear capacity curve for the plate representing the remaining capacity of plate when reduced for global in-plane membrane stress (longitudinal, transverse and shear stresses).

The design bending stress for stiffener flange side is given as a linear capacity curve for the stiffener flange representing the remaining capacity of stiffener when reduced for global longitudinal membrane stress.

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**402** The requirement in 401 applies to an axis parallel to the plating. For stiffeners at an oblique angle with the plating an approximate requirement to standard section modulus may be obtained by multiplying the section modulus from 401 with the factor:

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

$\alpha$ = 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:

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

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 shall not 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 shall be carried out.

**303** The section modulus and web area of the girder shall be taken in accordance with particulars as given in 400 and 500. 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 $b_e$ is determined by the following formula:

$$b_e = C_e b$$

$C_e$ = parameter given in Fig.1 for various numbers of evenly spaced point loads ($N_p$) on the girder span
b = full breadth of plate flange (m), e.g. span of the supported stiffeners, or distance between girders, see also 602.
$l_0$ = 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.

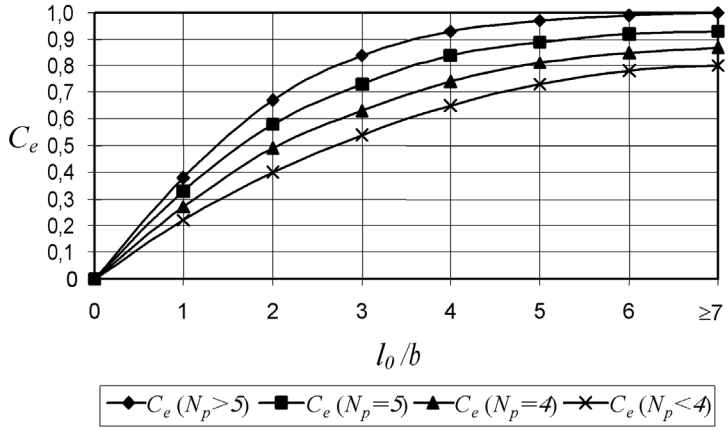



**Figure 1**
**Graphs for the effective flange parameter $C_e$**

## 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:

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

**602** *Section modulus $Z_g$*

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

S = girder span (m). The web height of in-plane girders may be deducted. When brackets are fitted at the ends, the girder span S may be reduced by two thirds of the bracket arm length, provided the girder ends may be assumed clamped and provided the section modulus at the bracketed ends is satisfactory

b = breadth of load area (m) (plate flange), b may be determined as:
= 0.5 ($l_1 + l_2$), $l_1$ and $l_2$ are the spans of the supported stiffeners on both sides of the girder, respectively, or distance between girders

$p_d$ = design pressure (kN/m$^2$) as given in Sec.3

$k_m$ = bending moment factor, see 604

$\sigma_{pd2}$ = design bending stress (N/mm$^2$), see F401

$\sigma_{jd}$ = equivalent design stress for global in-plane membrane stress.

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

**603** *Web area $A_w$*

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

$k_\tau$ = shear force factor, see 604.

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

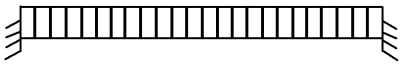
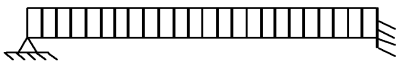
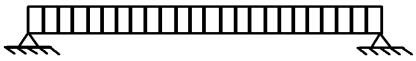
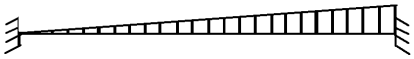
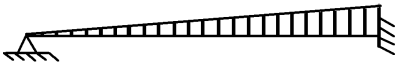
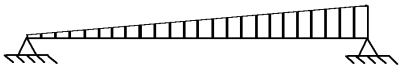
$N_p$ = number of supported stiffeners on the girder span

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

$\tau_{pd}$ = 0.5 $f_{yd}$ (N/mm$^2$).

**604** The $k_m$- and $k_\tau$-values referred to in 602 and 603 may be calculated according to general beam theory. In Table G1 $k_m$- and $k_\tau$-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 $k_\tau$**

| *Load and boundary conditions* | | | *Bending moment and shear force factors* | | |
|---|---|---|---|---|---|
| *Positions* | | | 1 | 2 | 3 |
| 1<br>Support | 2<br>Field | 3<br>Support | $k_{m1}$<br>$k_{\tau 1}$ | $k_{m2}$<br>- | $k_{m3}$<br>$k_{\tau 3}$ |
|  | | | 12<br>0.5 | 24 | 12<br>0.5 |
|  | | | 0.38 | 14.2 | 8<br>0.63 |
|  | | | 0.5 | 8 | 0.5 |
|  | | | 15<br>0.3 | 23.3 | 10<br>0.7 |
|  | | | 0.2 | 16.8 | 7.5<br>0.8 |
|  | | | 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
$\mu$ = friction coefficient
$\gamma_{Ms}$ = 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 $\mu$ is dependent on the specified class of surface treatment. The value of $\mu$ shall be taken according to Table H1.

**Table H1 Friction coefficient $\mu$**

| *Surface category* | $\mu$ |
|---|---|
| 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, see also DNV-OS-C401 Ch.2 Sec.6.

**110** 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,<br>- with any loose rust removed, no pitting<br>- spray metallised with aluminium<br>- 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*<br>*mm* | *Bolt diameter d (maximum)*<br>*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*<br>*mm* | *Bolt diameter d (maximum)*<br>*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*<br>*mm* | *Bolt diameter d (maximum)*<br>*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 of offshore structures. See also Sec.2 B105.

**102** The aim of fatigue design is to ensure that the structure has an adequate fatigue life. Calculated fatigue lives should also form the basis for 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, corresponding to the mean-minus-two-standard-deviation curves of relevant experimental data.

**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.

**107** The analyses shall be performed utilising relevant site specific environmental data for the area(s) in which the unit will be operated. The restrictions shall be described in the Operation Manual for the unit.

**108** For world wide operation the analyses shall be performed utilising environmental data (e.g. scatter diagram, spectrum) given in DNV-RP-C205. The North Atlantic scatter diagram shall be utilised.

### A 200 Design fatigue factors

**201** Design fatigue factors (DFF) shall be applied to reduce the probability for 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 may 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 satisfies the requirement to damaged condition according to the ALS with failure in the actual element 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:**

Units intended to follow normal inspection schedule according to class requirements, i.e. the 5-yearly inspection interval in sheltered waters or drydock, may apply a Design Fatigue Factor (DFF) of 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 (see splash zone definition in Sec.10 B200).

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 beneath positions 150 m below water level should be assumed inaccessible for in-service inspection.

**208** The various 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 below the acceptable limit, 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 $\gamma_M$ 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.

**Guidance note:**

Recommendations for design of structures exposed to accidental events can be found in DNV-RP-C204.

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**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 $\delta_{max}$* | *Limit for $\delta_2$* |
|---|---|---|
| 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:

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

$\delta_{max}$ = the sagging in the final state relative to the straight line joining the supports
$\delta_0$ = the pre-camber
$\delta_1$ = the variation of the deflection of the beam due to the permanent loads immediately after loading
$\delta_2$ = the variation of the deflection of the beam due to the variable loading plus any time dependent deformations due to the permanent load

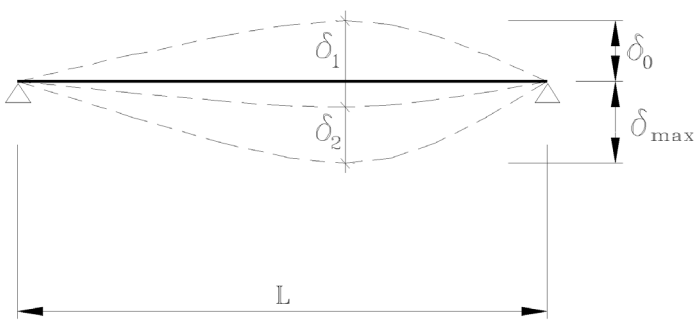


**Figure 1**
**Definitions of vertical deflections**

**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.

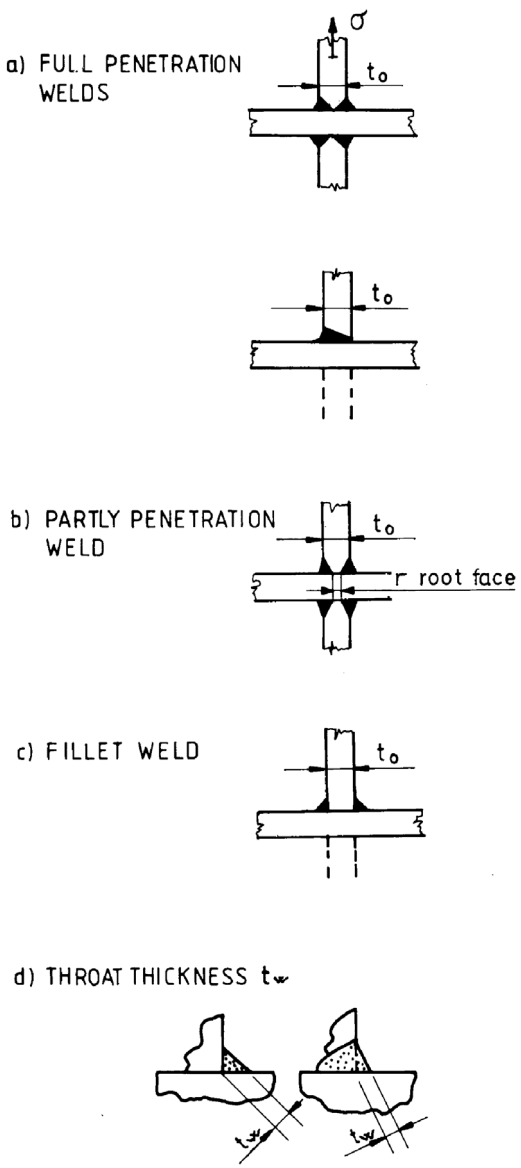


**Figure 1**
**Tee and cross joints**

**203** The type of connection shall 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 welds with abutting plate panels forming boundaries to open sea.
— All external welds in way of opening to open sea e.g. pipes, seachests or tee-joints.

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.

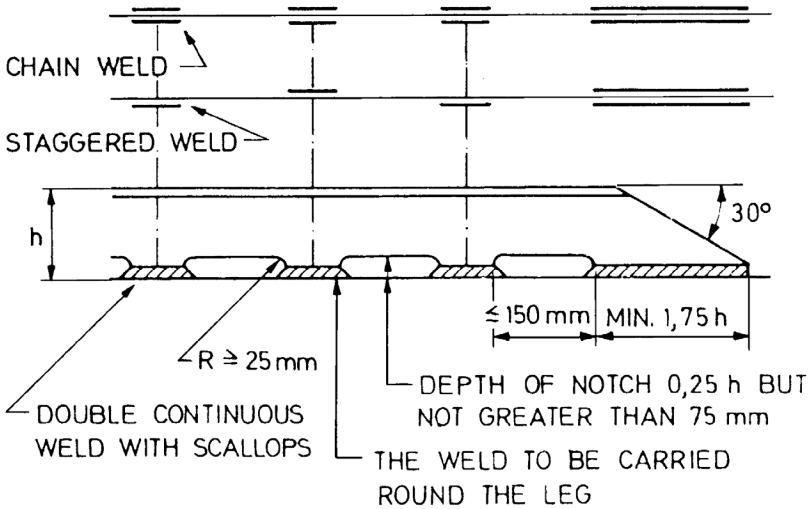


**Figure 2**
**Intermittent welds**

## 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.

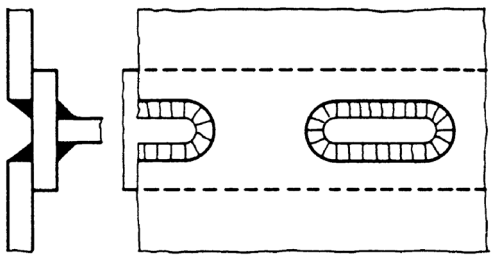

**Figure 3**
**Slot welds**

### 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 $\gamma_{Mw}$ for welded connections are given in Table C1.

| **Table C1 Material factors $\gamma_{Mw}$ 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 $\sigma_{fw}$ 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$^2$)
$\sigma_{fw}$ = 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 $\sigma_{fw}$ (N/mm$^2$) | Weld metal $f_w = \left(\frac{\sigma_{fw}}{235}\right)^{0,75}$ | Base metal/weld metal $f_r = \left(\frac{f_y}{\sigma_{fw}}\right)^{0,75}$ |
| Normal strength steels | NV NS | 355 | 1.36 | 0.75 |
| High strength steels | NV 27 | 375 | 1.42 | 0.75 |
| | NV 32 | 375 | 1.42 | 0.88 |
| | NV 36 | 375 | 1.42 | 0.96 |
| | NV 40 | 390 | 1.46 | 1.00 |

## 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:

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

$f_r$ = 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, $t_0$ need normally 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:

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

$t_0$ = net thickness abutting plate.

— for staggered welds:

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

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 design tensile stresses higher than 120 N/mm$^2$, the throat thickness of a double continuous weld is not to be less than:

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

minimum 3 mm.

$f_w$ = strength ratio as defined in 104

$\sigma_d$ = 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.

**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.

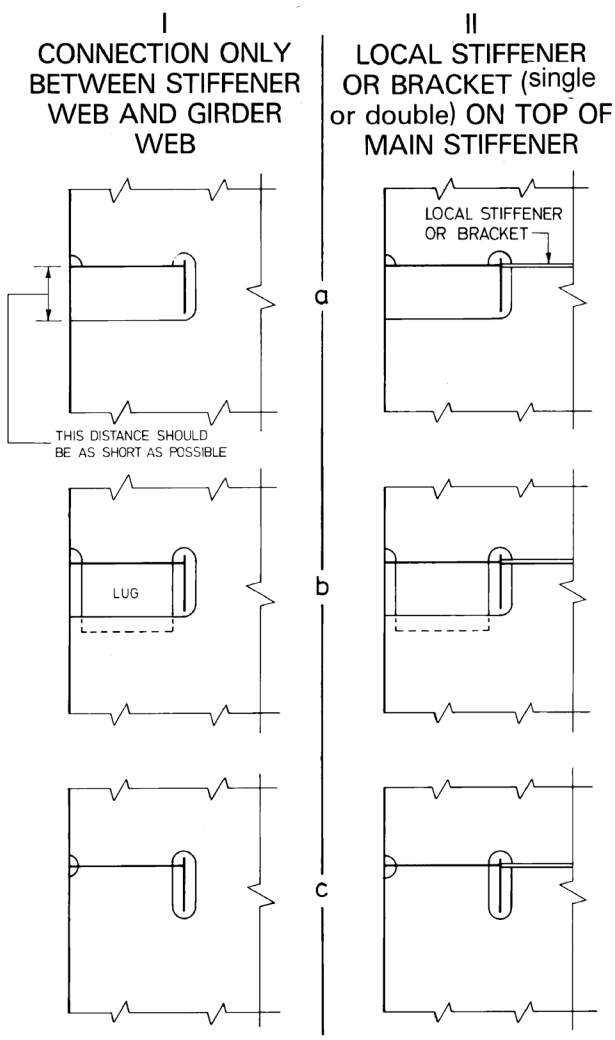


**Figure 5**
**Connections of stiffeners**

**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:

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

c = detail shape factor as given in Table C3

$f_{yd}$ = design yield strength $f_y/\gamma_M$.
$f_y$ is the minimum yield stress (N/mm$^2$) as given in Sec.4 Table D1
$l$ = span of stiffener (m)
s = distance between stiffeners (m)
$p_d$ = design pressure (kN/m$^2$).

**Table C3 Detail shape factor c**

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

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

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

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

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

$\tau_d$ = design shear stress in web plate (N/mm$^2$)
$f_w$ = strength ratio for weld as defined in 104
$f_r$ = strength ratio 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:

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

and

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

$\sigma_{\perp d}$ = 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
$\beta_w$ = appropriate correlation factor, see Table C4
$\gamma_{Mw}$ = material factor for welds

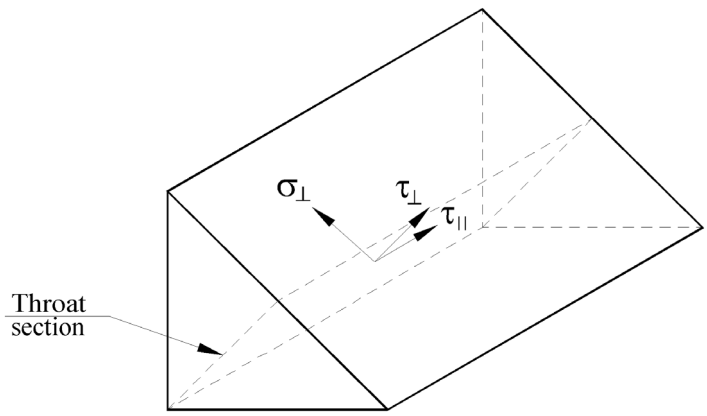


**Figure 6**
**Stresses in fillet weld**

| **Table C4 The correlation factor $\beta$w** | | |
|---|---|---|
| *Steel grade* | *Lowest ultimate tensile strength* $f_u$ | *Correlation factor* $\beta_w$ |
| 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 CONTROL

## A. General

### A 100 General

**101** Corrosion control of structural steel for offshore structures comprises:

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

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

## B. Techniques for Corrosion Control Related to Environmental Zones

### B 100 Atmospheric zone

**101** Steel surfaces in the atmospheric zone shall be protected by a coating system (see D100) proven for marine atmospheres by practical experience or relevant testing.

**Guidance note:**

The "Atmospheric Zone" is defined as the areas of a structure above the Splash Zone (see B201) being exposed to sea spray, atmospheric precipitation and/or condensation.

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### B 200 Splash zone

**201** Steel surfaces in the splash zone shall be protected by a coating system (see D100) proven for splash zone applications by practical experience or relevant testing. A corrosion allowance should also be considered in combination with a coating system for especially critical structural items.

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

**203** 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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**204** For floating units with constant draught, the extent of the splash zone shall extend 5 m above and 4 m below this draught.

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

The wave height to be used to determine the upper and lower limits of the splash zone shall be taken as 1/3 of the wave height that has an annual probability of being exceeded of $10^{-2}$.

**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 shall have a cathodic protection system. The cathodic protection design shall include current drain to any electrically connected items for which cathodic protection is not considered necessary (e.g. piles).

The cathodic protection shall also include the splash zone beneath MSL (for bottom fixed structures) and splash zone beneath normal operating draught (for floating units), see B202.

**Guidance note:**

The 'Submerged Zone' is defined as the zone below the splash zone.

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

**Guidance note:**

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

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### B 400 Internal zones

**401** Internal zones exposed to seawater for a main period of time (e.g. ballast tanks) shall be protected by a coating system (see D100) proven for such applications by practical experience or relevant testing. Cathodic protection should be considered for use in combination with coating (see also 402).

**Guidance note:**

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

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

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

**404** To facilitate inspection, light coloured and hard coatings shall be used for components of internal zones subject to major fatigue forces requiring visual inspection for cracks. Regarding restrictions for use of coatings

with high content of aluminium, see D101.

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

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

**407** A corrosion allowance shall be implemented for internal compartments without any corrosion protection (coating and/or cathodic protection) but subject to a potentially corrosive environment such as intermittent exposure to seawater, humid atmosphere or produced/cargo oil.

Any corrosion allowance for individual components (e.g. plates, stiffeners and girders) shall be defined taking into account:

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

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

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

## C. Cathodic Protection

### C 100 General

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

**Guidance note:**

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

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

**Guidance note:**

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

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

**Guidance note:**

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

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**104** Cathodic protection may cause hydrogen induced stress cracking (HISC) of components in high strength steels that are exposed to severe straining in service

It is recommended that the welding of high strength structural steels is qualified to limit the hardness in the weld zone to max. 350 HV (Vicker hardness). The use of coatings reduces the risk of hydrogen embrittlement further and is recommended for all critical components in high strength structural steel.

**Guidance note:**

There is no evidence in the literature that structural steels with SMYS up to 550 N/mm$^2$ have suffered any cracking when exposed to cathodic protection in marine environments at the protection potential range given in 102.

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### C 200 Galvanic anode systems

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

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

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

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

**204** Requirements to the manufacturing of anodes (see 205) shall be defined during design, e.g. by reference to a standard or in a project specification.

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

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

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

**207** Manufacturing and installation of galvanic anodes are addressed in DNV-OS-C401, Sec.5

### C 300 Impressed current systems

**301** Impressed current anodes and reference electrodes for control of current output shall be designed with a design life at least equal to that of the offshore installation unless replacement of anodes (and other critical components) during operation is presumed. It is recommended that the design in any case allows for replacement of any defective anodes and reference electrodes (see 304) during operation.

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

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

**Guidance note:**

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

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

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

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

— reference to design code and design premises
— calculations of surface areas and cathodic current demand (mean and initial/final) for individual sections

of the structure
— general arrangement drawings showing locations of anodes, anode shields, reference electrodes, cables and rectifiers
— detailed drawings of anodes, reference electrodes and other major components of the system
— documentation of anode and reference electrode performance to justify the specified design life
— documentation of rectifiers and current control system
— documentation of sizing of anode shields
— specification of anode shield materials and application
— commissioning procedure, incl. verification of proper protection range by independent potential measurements
— operational manual, including procedures for replacement of anodes and reference electrodes.

**307** Manufacturing and installation of impressed current cathodic protection systems are addressed in DNV-OS-C401, Sec.5

## D. Coating systems

### D 100 Specification of coating

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

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

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

**Guidance note:**

It is recommended that supplier specific coating materials are qualified by relevant testing or documented *performance* in service.

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**102** Coating materials and application of coatings are addressed in DNV-OS-C401, Sec.5.

# 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-structure interaction.

The foundation structure itself (anchor), including the anchor padeye, shall be designed for the loads and acceptance criteria specified in DNV-OS-E301 Ch.2 Sec.4. See also design requirements in this standard Sec.4 to Sec.10, as relevant for steel anchor design.

**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 ($\gamma_M$)
— for total shear stress analysis, the characteristic undrained shear strength shall be divided by the material coefficient ($\gamma_M$).

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 elaborations 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-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-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 ($\gamma_M$):

$\gamma_M$ = 1.2 for effective stress analysis
= 1.3 for total stress analysis.

**104** For ALS design the material coefficients $\gamma_M$ 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 for selection of representative loads.

**106** For determination of design soil resistance against axial pile loads in ULS design, a material coefficient $\gamma_M = 1.3$ 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 $\gamma_M = 1.7$ 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:

$\gamma_M$ = 1.2 for effective stress analysis
= 1.3 for total stress analysis.

**109** For ALS and SLS design, the material coefficient $\gamma_M$ 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 $\gamma_M = 1.0$.

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 $\gamma_M = 1.0$.

### 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:

$\gamma_M$ = 1.2 for effective stress analysis
= 1.3 for total stress analysis.

For ALS design $\gamma_M = 1.0$ 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 (300)
— gravity anchors (400)
— suction anchors (500)
— fluke anchors (600)
— plate anchors (700).

**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:

*Consequence class 1 (CC1):* Failure is unlikely to lead to unacceptable consequences such as loss of life, collision with an adjacent platform, uncontrolled outflow of oil or gas, capsize or sinking.

*Consequence class 2 (CC2):* 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 (i.e. water depth exceeding 200m, see DNV-OS-E301 Ch.2 Sec.2 B100) a dynamic analysis is required.

**Table E1 Load coefficients for ULS 1)**

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

1) If the characteristic mean tension exceeds 2/3 of the characteristic dynamic tension, when applying a dynamic analysis in ULS consequence class 1, then a common value of 1.3 shall be applied on the characteristic tension instead of the partial load factors given in Table E1, ref. DNV-OS-E301. This is intended to ensure adequate safety in cases dominated by a mean tension component. The partial safety factor on the characteristic anchor resistance given in Table E1 is applicable in such cases provided that the effects of creep and drainage on the shear strength under the long-term load are accounted for.

**Table E2 Load coefficients for ALS**

| *Consequence class* | *Type of analysis* | $\gamma_{mean}$ | $\gamma_{dyn}$ |
|---|---|---|---|
| 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 $\gamma_{mean}$ and $\gamma_{dyn}$, 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 700.

### E 300 Pile anchors

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

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

$\gamma_M$ = 1.3 for ULS Consequence Class 1 (CC1) and 2 (CC2)
= 1.0 for ALS CC1 and CC2.

See also requirements to tension piles in C107.

### E 400 Gravity anchors

**401** 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.

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

$\gamma_M$ = 1.3 for ULS Consequence Class 1 (CC1) and 2 (CC2)
= 1.0 for ALS CC1 and CC2.

**E 500 Suction anchors**

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

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

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

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

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

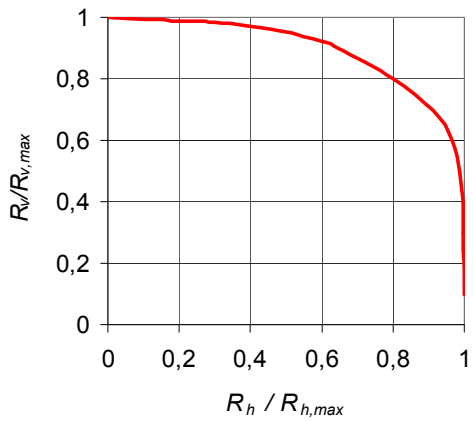



**Figure 1**
**Schematic resistance diagram for suction anchor.**

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

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

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

$\gamma_M$ = 1.20 for ULS Consequence Class 1 (CC1) and 2 (CC2)
= 1.20 for ALS CC2, and
= 1.00 for ALS CC1

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

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

**509** Load factors for loads associated with impact landing, suction to target penetration depth and possible retrieval by means of overpressure shall be taken according to Sec.2 D400. For loads associated with permanent removal after service life the load factors may be taken according to Sec.2 D700.

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

### E 600 Fluke anchors

**601** 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.

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

**603** 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.

**604** 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.

**605** Provided that the uncertainty in the load measurements is accounted for and that the target installation tension $T_i$ is reached and verified by reliable measurements the main uncertainty in the anchor resistance lies then in the predicted post-installation effects mentioned above.

The soil material coefficient $\gamma_M$ on this predicted component of the anchor resistance shall then be:

$\gamma_M$ = 1.3 for ULS Consequence Class 1 (CC1) and 2 (CC2)
= 1.0 for ALS CC1
= 1.3 for ALS CC2.

### E 700 Plate anchors

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

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

# 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 |

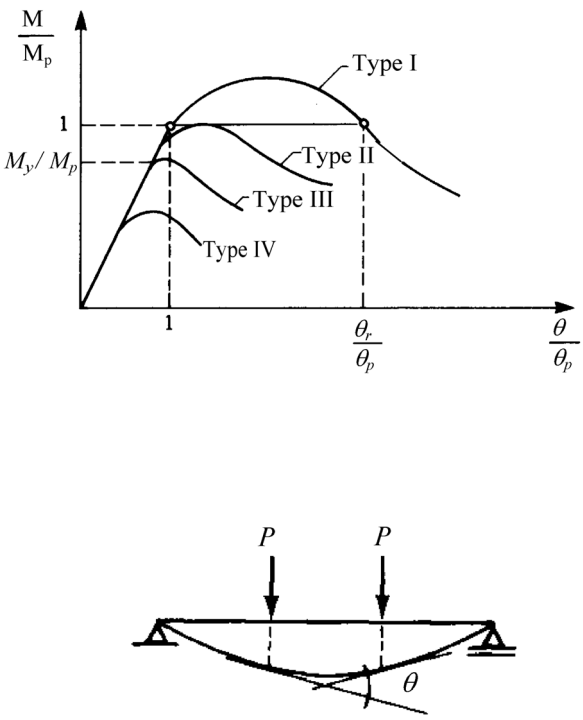



**Figure 1**
**Relation between moment M and plastic moment resistance $M_p$, and rotation $\theta$ for cross sectional types. $M_y$ is elastic moment resistance**

**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**

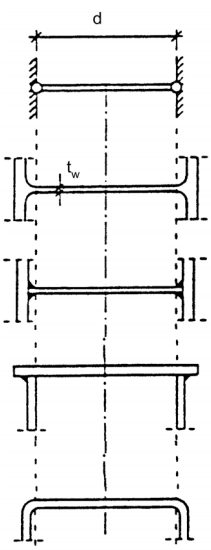
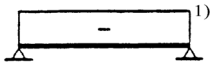
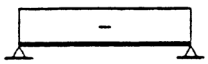
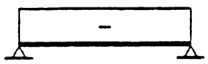
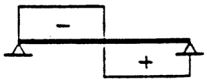
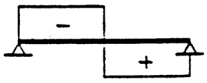
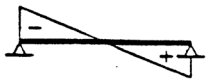
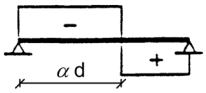
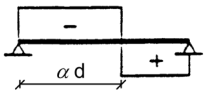
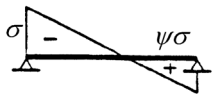
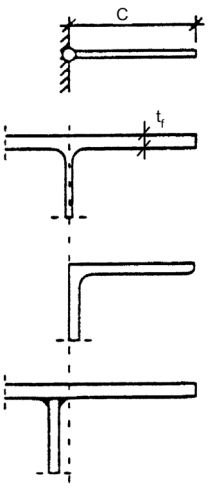
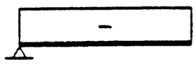
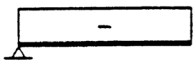
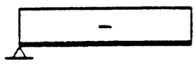
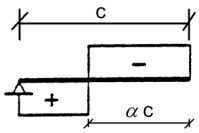
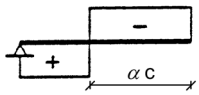
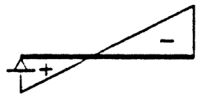
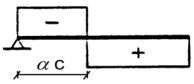
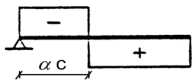
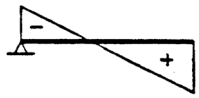
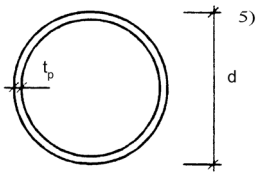
| *NV Steel grade* 1) | $\varepsilon$ 2) |
|---|---|
| NV-NS | 1 |
| NV-27 | 0.94 |
| NV-32 | 0.86 |
| NV-36 | 0.81 |
| NV-40 | 0.78 |
| NV-420 | 0.75 |
| NV-460 | 0.72 |
| NV-500 | 0.69 |
| NV-550 | 0.65 |
| NV-620 | 0.62 |
| NV-690 | 0.58 |

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

2)

$$\varepsilon = \sqrt{\frac{235}{f_y}} \quad \text{where } f_y \text{ is yield strength}$$

**Table A3 Maximum width to thickness ratios for compression elements**

| *Cross section part* | *Type I* | *Type II* | *Type III* |
|---|---|---|---|
|  $d = h - 3\,t_w$ 3) |  1) $d / t_w \le 33\,\varepsilon$ 2) |  $d / t_w \le 38\,\varepsilon$ |  $d / t_w \le 42\,\varepsilon$ |
| |  $d / t_w \le 72\,\varepsilon$ |  $d / t_w \le 83\,\varepsilon$ |  $d / t_w \le 124\,\varepsilon$ |
| |  when $\alpha > 0.5$: $d/t_w \le \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0.5$: $d/t_w \le \frac{36\varepsilon}{\alpha}$ |  when $\alpha > 0.5$: $d/t_w \le \frac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0.5$: $d/t_w \le \frac{41.5\varepsilon}{\alpha}$ |  when $\psi > -1$: $d/t_w \le \frac{126\varepsilon}{2 + \psi}$ 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: $c/t_f \le 14\varepsilon$ |
| | Tip in compression  Rolled: $c/t_f \le 10\varepsilon/\alpha$ Welded: $c/t_f \le 9\varepsilon/\alpha$ | Tip in compression  Rolled: $c/t_f \le 11\varepsilon/\alpha$ Welded: $c/t_f \le 10\varepsilon/\alpha$ | Tip in compression  4) Rolled: $c/t_f \le 23\varepsilon\sqrt{C}$ Welded: $c/t_f \le 21\varepsilon\sqrt{C}$ |
| | Tip in tension  Rolled: $c/t_f \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$ Welded: $c/t_f \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$ | Tip in tension  Rolled: $c/t_f \le \frac{11\varepsilon}{\alpha\sqrt{\alpha}}$ Welded: $c/t_f \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$ | Tip in tension  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) $\varepsilon$ is defined in Table A2
3) Valid for rectangular hollow sections (RHS) where h is the height of the profile
4) C is the buckling coefficient. See Eurocode 3 Table 5.3.3 (denoted $k_\sigma$)
5) Valid for axial and bending, not external pressure.