

STANDARD

DNVGL-ST-0126

Edition July 2018

Support structures for wind turbines

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FOREWORD

DNV GL standards contain requirements, principles and acceptance criteria for objects, personnel, organisations and/or operations.

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CHANGES – CURRENT

This document supersedes the April 2016 edition of DNVGL-ST-0126.

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

Changes July 2018

<i>Topic</i>	<i>Reference</i>	<i>Description</i>
Target safety level	[2.3.1.5]	New text regarding the target safety level for the standard has been included.
Additional site conditions	[3.2.2]	Additional site conditions for offshore wind turbines which were also in DNV-OS-J101 has been included.
Geometrical tolerances and settlements	[3.5.3], [3.10.2], [4.5.1.8]	The text regarding load impact from geometrical tolerances and settlements has been updated.
Service vessel impact	[3.7.1], [3.9.1.4], [4.14.2.15], [5.6.5]	The requirements for design against service vessel impact has been updated in accordance with DNVGL-ST-0437 and the new draft IEC standard for fixed offshore wind turbines, IEC 61400-3-1.
Transportation and installation loads	[3.13]	Reference to DNVGL-ST-0054 and DNVGL-ST-N001 has been included regarding transportation and installation loads.
Selection of steel material	[4.2.2]	The text regarding temperatures to be applied for selection of steel grades has been corrected.
Structural steel thickness limitations	[4.2.5]	The thickness limitations have been aligned with DNVGL-OS-C101.
Buckling analysis	[4.4], [4.5], [4.6]	The text regarding buckling has been updated.
Slip resistance for preloaded bolts	[4.9.5.5]	The text regarding slip resistance for a preloaded bolt has been replaced by a reference to DNVGL-OS-C101.
Multi-axially stressed welded connections	[4.11.3.2]	Guidance note regarding multi-axially stressed welded connections has been included.
Steel fatigue crack closure	[4.11.3.12]	The text regarding fatigue crack closure has been replaced by a reference to DNVGL-RP-C203.

<i>Topic</i>	<i>Reference</i>	<i>Description</i>
Steel S-N curves and design fatigue factors	[4.11.5]	Selection of design fatigue factors, DFFs, has been updated and the splash zone divided in upper and lower splash zone. Further, more guidance text regarding S-N curve selection for grinded welds has been introduced.
Alignment with DNVGL-ST-C502	Sec.5, Sec.6, App.C	Sec.5, Sec.6 and App.C have been updated based on the publication of DNVGL-ST-C502 and the changes in that document compared to its precursor DNV-OS-C502.
Decompression underneath tower bottom flange	[5.9.2.1]	New text regarding decompression underneath the tower bottom flange has been included.
Alignment with DNVGL-RP-C212	Sec.7	Sec.7 has been updated based on the publication of DNVGL-RP-C212 and the changes in that document compared to its precursor DNV Classification Notes 30.4.
Characteristic geotechnical parameters	[7.4.2]	The paragraph on characteristic geotechnical parameters has been reformulated.
Inspection of steel structures	[9.3.2.2]	More specific requirements included for the required inspections of steel structures when reduced detailed design factors have been applied in the design.
Periodical inspections for scour protection	[9.6]	The need for periodical inspections for scour protection has been specified in a separate paragraph.
Geotechnical calculations	N/A	App.F and App.G have been deleted due to issuance of DNVGL-RP-C212.

Editorial corrections

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



Acknowledgements

This standard makes use of two figures provided by Mærsk Olie og Gas A/S (Total S.A.). The two figures are [Figure 4-5](#) and [Figure 4-6](#). Mærsk Olie og Gas A/S (Total S.A.) is gratefully acknowledged for granting DNV GL permission to use this material.

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SECTION 1 GENERAL

1.1 Introduction

This document constitutes the DNV GL standard for design of wind turbine support structures and the development has been based on long experience in DNV GL with issuing standards to help the wind turbine industry moving forward. The standard takes construction, transportation, installation and inspection issues into account to the extent necessary in the context of structural design. The design principles and overall requirements are defined in the standard. The standard shall be used together with other relevant standards as listed in [1.5].

1.2 Objective

The standard specifies general principles and guidelines for the structural design of wind turbine support structures.

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 referenced standards, recommended practices, guidelines, etc.)
- serve as a contractual reference document between suppliers and purchasers related to design, construction, installation and in-service inspection
- serve as a guideline for designers, suppliers, purchasers and regulators
- specify procedures and requirements for support structures subject to DNV GL certification
- serve as a basis for verification of wind turbine support structures for which DNV GL is contracted to perform the verification and certification.

1.3 Scope

This DNV GL standard for wind turbine support structures offers pragmatic design approaches and can for example be used for design of steel and concrete towers, gravity-based concrete foundations, together with steel foundations such as monopile foundations, jacket structure foundations and suction buckets foundations.

This standard gives requirements for the following:

- design principles
- selection of material and extent of inspection
- selection of design loads and load combinations
- load effect analyses
- structural design
- corrosion protection
- geotechnical design
- grouted connections for offshore support structures
- scour protection for offshore support structures.

The standard also contains requirements for materials, execution, operations and maintenance (O&M) related to the design. This is done by inclusion of for example references to material and execution standards.

1.4 Application

The standard is applicable to all types of onshore and fixed offshore support structures for wind turbines.

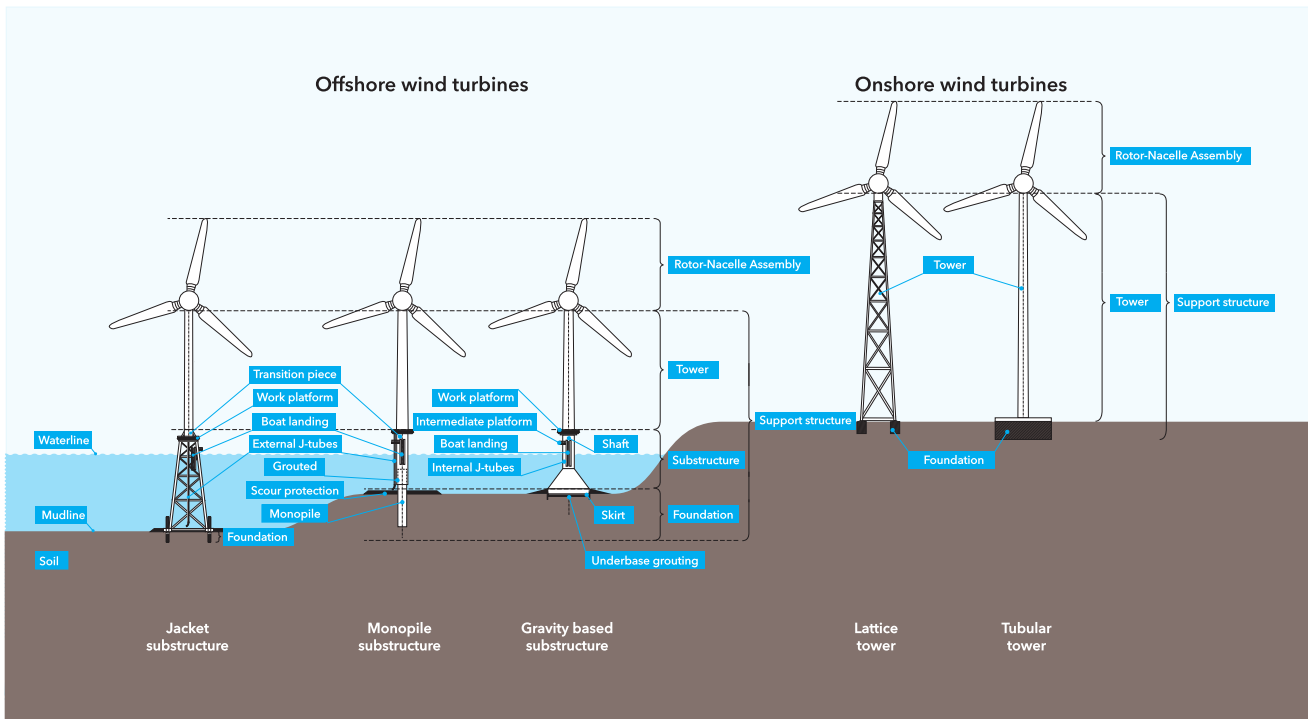


Figure 1-1 Definition of wind turbine components

The standard has been written for worldwide application. National and governmental regulations may include requirements in excess of the provisions given by this standard depending on the size, type, location and intended service of the wind turbine structure.

Guidance note:

The present DNV GL standard covers the technical requirements to be applied within the DNV GL certification schemes according to DNVGL-SE-0073, DNVGL-SE-0074 and DNVGL-SE-0190, and it is also intended to cover the requirements implied when using IEC 61400-22 related certification schemes.

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The standard is applicable to the design of complete structures, including towers, substructures and foundations, but excluding wind turbine components such as nacelles and rotors.

For wind turbine support structures a coherent set of standards for materials, design and construction shall be applied and supplemented by the provisions in this standard.

The standard does not cover design of support structures for substations for wind farms. For design of support structures for substations, such as converter stations and transformer stations, DNVGL-ST-0145 applies.

The standard may also be used for design of support structures for other structures in an offshore wind farm, such as meteorological masts (DNVGL-SE-0420).

For design of floating wind turbine structures and their station keeping systems DNVGL-ST-0119 applies.

The standard does not cover design of wind turbine components such as nacelle, rotor, generator and gearbox. For structural design of rotor blades DNVGL-ST-0376 applies. For structural design of wind turbine components DNVGL-ST-0361 applies.

Design loads and load combinations to be considered together with this standard shall be determined as described in DNVGL-ST-0437 and [Sec.3](#).

1.5 References

1.5.1 General

1.5.1.1 The standards and guidelines in [Table 1-1](#) and [Table 1-2](#) include provisions which, through reference in the text constitute requirements of this standard. See also current list of DNV GL service documents <http://rules.dnvgl.com>.

1.5.1.2 References are either normative or informative. Normative references in this document are indispensable for its application. Informative references provide additional information intended to assist the understanding or use of the document.

Guidance note:

Normative references are typically referred to as "testing shall be performed in accordance with ISO xxx", while informative references are typically referred to as "testing may be performed in accordance with ISO xxx or ISO yyyy", or "for testing, see DNVGL-RP-xxxx".

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Table 1-1 DNV GL documents

<i>Document code</i>	<i>Title</i>
DNV Classification Notes 30.6	Structural Reliability Analysis of Marine Structures
DNV-RP-C201	Buckling Strength of Plated Structures
DNVGL-CG-0051	Non-destructive testing
DNVGL-OS-B101	Metallic materials
DNVGL-OS-C101	Design of offshore steel structures, general - LRFD method
DNVGL-OS-C401	Fabrication and testing of offshore structures
DNVGL-RP-0360	Subsea power cables in shallow water
DNVGL-RP-0416	Corrosion protection for wind turbines
DNVGL-RP-0419	Analysis of grouted connections using the finite element method
DNVGL-RP-A203	Technology qualification
DNVGL-RP-C202	Buckling strength of shells
DNVGL-RP-C203	Fatigue design of offshore steel structures
DNVGL-RP-C204	Design against accidental loads
DNVGL-RP-C205	Environmental conditions and environmental loads
DNVGL-RP-C207	Statistical representation of soil data
DNVGL-RP-C208	Determination of structural capacity by non-linear finite element analysis methods
DNVGL-RP-C210	Probabilistic methods for planning of inspection for fatigue cracks in offshore structures
DNVGL-RP-C212	Offshore soil mechanics and geotechnical engineering
DNVGL-SE-0073	Project certification of wind farms according to IEC 61400-22

<i>Document code</i>	<i>Title</i>
DNVGL-SE-0074	Type and component certification of wind turbines according to IEC 61400-22
DNVGL-SE-0190	Project certification of wind power plants
DNVGL-SE-0420	Certification of meteorological masts
DNVGL-ST-0054	Transport and installation of wind power plants
DNVGL-ST-0119	Floating wind turbine structures
DNVGL-ST-0145	Offshore substations
DNVGL-ST-0359	Subsea power cables for wind power plants
DNVGL-ST-0361	Machinery for wind turbines
DNVGL-ST-0376	Rotor blades for wind turbines
DNVGL-ST-0437	Loads and site conditions for wind turbines
DNVGL-ST-C502	Offshore concrete structures
DNVGL-ST-N001	Marine operations and marine warranty

Table 1-2 External documents

<i>Document code</i>	<i>Title</i>
API RP 2X	Recommended Practice for Ultrasonic and Magnetic Examination of Offshore Structural Fabrication and Guidelines for Qualification of Technicians
ASTM C348	Standard Test Method for Flexural Strength of Hydraulic-Cement Mortars
BS 1881-122:2011	Testing concrete. Method for determination of water absorption
BS 7910	Guide on methods for assessing the acceptability of flaws in fusion welded structures
BSH 7005	Standard Konstruktion. Mindestanforderungen an die konstruktive Ausführung von Offshore-Bauwerken in der ausschließlichen Wirtschaftszone (AWZ)
DIBt:2012	Richtlinie für Windenergieanlagen, Einwirkungen und Standsicherheitsnachweise für Turm und Gründung, Fassung Oktober 2012
EN 196-1	Methods of testing cement - Part 1: Determination of strength
EN 197-1	Cement - Part 1: Composition, specifications and conformity criteria for common cements
EN 197-2	Cement - Part 2: Conformity evaluation
EN 206	Concrete – Specification, performance, production and conformity
EN 1090-1	Execution of steel structures and aluminium structures – Part 1: Requirements for conformity assessment of structural components
EN 1090-2	Execution of steel structures and aluminium structures - Part 2: Technical requirements for steel structures
EN 1990	Eurocode – Basis of structural design
EN 1992-1-1	Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings

<i>Document code</i>	<i>Title</i>
EN 1992-2	Eurocode 2: Design of concrete structures – Concrete bridges – Design and detailing rules
EN 1993-1-1	Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings
EN 1993-1-5	Eurocode 3: Design of steel structures - Part 1-5: Plated structural elements
EN 1993-1-6	Eurocode 3: Design of Steel Structures, Part 1-6: Strength and Stability of Shell Structures
EN 1993-1-7	Eurocode 3: Design of steel structures - Part 1-7: Plated structures subject to out of plane loading
EN 1993-1-8	Eurocode 3: Design of steel structures - Part 1-8: Design of joints
EN 1993-1-9	Eurocode 3: Design of steel structures - Part 1-9: Fatigue
EN 1993-1-10	Eurocode 3: Design of steel structures - Part 1-10: Material toughness and through-thickness properties
EN 1993-2	Eurocode 3: Design of steel structures - Part 2: Steel Bridges
EN 1993-3-1	Eurocode 3: Design of steel structures - Part 3-1: Towers, masts and chimneys - Towers and masts
EN 1997-1	Eurocode 7: Geotechnical design – Part 1: General Rules
EN 1997-2	Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing
EN 1998-5	Eurocode 8: Design of structures for earthquake resistance - Part 5: Foundations, retaining structures and geotechnical aspects
EN 10025-1	Hot rolled products of non-alloy structural steels
EN 10025-2	Hot rolled products of structural steels. Technical delivery conditions for non-alloy structural steels
EN 10025-3	Hot rolled products of structural steels. Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels
EN 10025-4	Hot rolled products of structural steels. Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels
EN 10080	Steel for the reinforcement of concrete - Weldable reinforcing steel - General
EN 10088-1	Stainless steels - Part 1: List of stainless steels
EN 10138	Prestressing steels
EN 10204	Metallic products – types of inspection documents
EN 10225	Weldable structural steels for fixed offshore structures – technical delivery conditions
EN 10228-3	Non-destructive testing of steel forgings - Part 3: Ultrasonic testing of ferritic and martensitic steel forgings
EN 12495	Corrosion Protection of Fixed Offshore Structures
EN 13670	Execution of Concrete Structures – Part 1: Common rules
EN 14399 (All parts)	High-strength structural bolting assemblies for preloading
EN 50308	Wind Turbines – Protective measures – Requirements for design, operation and maintenance

<i>Document code</i>	<i>Title</i>
IEC61400-1	Wind Turbines – Part 1: Design Requirements
IEC61400-3	Wind Turbines – Part 3: Design requirements for offshore wind turbines
IEC61400-22	Wind Turbines – Part 22: Conformity testing and certification of wind turbines
IIW-1823-07 ex XIII-2151r4-07/ XV-1254r4-07	Recommendations for Fatigue Design of Welded Joints and Components, International Institute of Welding (IIW/IIS), December 2008
ISO 1461	Hot dip galvanized coatings on fabricated iron and steel articles – Specifications and test methods
ISO 12944 (all parts)	Paints and varnishes – Corrosion protection of steel structures by protective paint systems
ISO 19901-2	Seismic design procedures and criteria
ISO 19901-8	Petroleum and Natural Gas Industries – Specific Requirements for Offshore Structures – Part 8: Marine Soil Investigations
ISO 19902	Petroleum and Natural Gas Industries – Fixed Steel Offshore Structures
ISO 22475-1	Geotechnical investigation and testing – Sampling methods and groundwater measurements – Part 1: Technical principles for execution
Model Code 2010	fib Model Code for Concrete Structures 2010
NACE RP0176	Corrosion Control of Steel Offshore Platforms Associated with Petroleum Production
NORSOK M-501	Surface preparation and protective coating
NORSOK N-004	Design of Steel Structures
NS 3473:2003	Prosjektering av betongkonstruksjoner. Beregnings- og konstruksjonsregler.

Table 1-3 References to literature

<i>Reference</i>	<i>Title</i>
/1/	Petersen C. Stahlbau: Grundlagen der Berechnung und baulichen Ausbildung von Stahlbauten, 3. Auflage. Vieweg Verlag. Braunschweig/Wiesbaden. 1993
/2/	Seidel M. Zur Bemessung geschraubter Ringflanschverbindungen von Windenergieanlagen. Schriftenreihe des Instituts für Stahlbau der Leibniz Universität Hannover (Heft 20). Shaker Verlag, Aachen. 2001
/3/	Schmidt H, Neuper M. Zum elastostatischen Tragverhalten exzentrisch gezogener L-Stöße mit vorgespannten Schrauben. Verlag Ernst & Sohn. Stahlbau 66 (1997) Vol. 3, pages 163-168

1.6 Definitions and abbreviations

1.6.1 Definition of verbal forms

Table 1-4 Definition of verbal forms

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

1.6.2 Definition of terms

Table 1-5 Definition of terms

<i>Term</i>	<i>Explanation</i>
abnormal load	wind load resulting from one of a number of severe fault situations for the wind turbine, which result in activation of system protection functions Abnormal wind loads are in general less likely to occur than loads from any of the normal wind load cases considered for the ULS.
accidental limit states (ALS)	ensure that the structure resists accidental loads and maintain integrity and performance of the structure due to local damage or flooding
atmospheric zone	external region exposed to atmospheric conditions
basic design standard	standard(s) from Table 1-1 selected as a basis for the design in combination with this standard
blade transition frequency	rotating frequency of the rotor multiplied by the number of rotor blades
cathodic protection	technique to prevent corrosion of a steel surface by making the surface to be the cathode of an electrochemical cell
characteristic load	reference value of a load to be used in the determination of the design load The characteristic load is normally based upon a defined quantile in the upper tail of the distribution function for load.
characteristic load effect	reference value of a load effect to be used in the determination of the design load effect The characteristic load effect is normally based upon a defined quantile in the upper tail of the distribution function for load effect.
characteristic material strength	nominal value of a material strength to be used in the determination of the design strength The characteristic material strength is normally based upon a 5% quantile in the lower tail of the distribution function for material strength.

<i>Term</i>	<i>Explanation</i>
characteristic resistance	reference value of a structural strength to be used in the determination of the design resistance The characteristic resistance is normally based upon a 5% quantile in the lower tail of the distribution function for resistance.
characteristic value	representative value of a load variable or a resistance variable For a load variable, it is a high but measurable value with a prescribed probability of not being unfavourably exceeded during some reference period. For a resistance variable it is a low but measurable value with a prescribed probability of being favourably exceeded.
coating	metallic, inorganic or organic material applied to steel surfaces for prevention of corrosion
construction	building and installing the support structure Also the terms fabrication and/or manufacturing is used for building a support structure.
contractor	party contractually appointed by the purchaser to fulfil all, or any of, the activities associated with fabrication and testing
corrosion allowance	extra wall thickness added during design to compensate for any reduction in wall thickness by corrosion (externally and internally) during design life
current	flow of water past a fixed point and usually represented by a velocity and a direction
design basis	document defining owner's requirements and conditions to be taken into account for design and in which any requirements in excess of this standard should be given
design life	period of time over which the structure in question is designed to provide an acceptable minimum level of safety, i.e. the period of time over which the structure is designed to meet the requirements set forth in this standard
design temperature	reference temperature for assessing areas where the unit can be transported, installed and operated The design temperature shall be lower or equal to the lowest mean daily temperature in air for the relevant areas. For seasonal restricted operations the lowest mean daily temperature in air for the season may be applied.
design value	value to be used in the deterministic design procedure, i.e. characteristic value modified by the resistance factor or the load factor, whichever is applicable
driving voltage	difference between closed circuit anode potential and protection potential
environmental state	short term condition of typically 10 minutes, 1 hour or 3 hours duration during which the intensities of environmental processes such as wave and wind processes can be assumed to be constant, i.e. the processes themselves are stationary
expected value	mean value, e.g. the mean value of a load during a specified time period
fatigue	degradation of the material caused by cyclic loading
fatigue critical	structure with predicted fatigue life near the design fatigue life
fatigue limit states (FLS)	possibility of failure due to the cumulative damage effect of cyclic loading
foundation	structural or geotechnical component, or both, extending from the ground level or the seabed and downwards See also Figure 1-1 .

<i>Term</i>	<i>Explanation</i>
general seabed level change	topographic changes which are not influenced by the presence of a structure (as opposed to global and local scour)
global scour	scour within the footprint of a structure, for example inside a jacket structure
highest astronomical tide (HAT)	level of high tide when all harmonic components causing the tide are in phase
hub height	height of centre of swept area of wind turbine rotor, measured from ground level onshore or mean water level (MWL) offshore
inspection	activities such as measuring, examination, testing, gauging one or more characteristics of an object or service and comparing the results with specified requirements to determine conformity
limit state	state beyond which the structure no longer satisfies the requirements The following categories of limit states are of relevance for structures: <ul style="list-style-type: none"> – ultimate limit states (ULS) – fatigue limit states (FLS) – accidental limit states (ALS) – serviceability limit states (SLS).
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.
local scour	scour around the structure, for example around one leg of a jacket structure or around a monopile structure
lower scour depth	smallest scour depth to be applied in the geotechnical and structural analysis
lowest astronomical tide (LAT)	level of low tide when all harmonic components causing the tide are in phase
lowest daily mean temperature	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: <ul style="list-style-type: none"> – Mean daily average temperature: the statistical mean average temperature for a specific calendar day. – Mean: statistical mean based on number of years of observations. – Average: average during one day and night.
lowest waterline	typical light ballast waterline for ships, transit waterline or inspection waterline for other types of units
mean	statistical mean over observation period.
mean water level (MWL)	mean level between highest astronomical tide and lowest astronomical tide.
non-destructive testing (NDT)	structural tests and inspection of welds by visual inspection, radiographic testing, ultrasonic testing, magnetic particle testing, penetrant testing and other non-destructive methods for revealing defects and irregularities
operating conditions	conditions wherein a unit is on location for purposes of drilling or other similar operations, and combined environmental and operational loadings are within the appropriate design limits established for such operations The unit may be either afloat or supported by the sea bed, as applicable.
partial safety factor method	method for design where uncertainties in loads are represented by a load factor and uncertainties in strengths are represented by a material factor

<i>Term</i>	<i>Explanation</i>
pile head	the position along a foundation pile in level with terrain or the unscoured seabed This definition applies regardless of whether the pile extends above the seabed.
pile length	length along a pile from pile head to pile tip
pile penetration	vertical distance from the seabed to the pile tip
potential	voltage between a submerged metal surface and a reference electrode
purchaser	owner or another party acting on his behalf, who is responsible for procuring materials, components or services intended for the design, construction or modification of a structure
redundancy	ability of a component or system to maintain or restore its function when a failure of a member or connection has occurred Redundancy can be achieved for instance by strengthening or introducing alternative load paths.
reference electrode	electrode with stable open-circuit potential used as reference for potential measurements
reliability	ability of a component or a system to perform its required function without failure during a specified time interval
risk	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.
rotor-nacelle assembly	part of wind turbine carried by the support structure
scour zone	external region of the unit which is located at the seabed and which is exposed to scour
service temperature	reference temperature on various structural parts of the unit used as a criterion for the selection of steel grades
serviceability limit states (SLS)	imply deformations in excess of tolerance without exceeding the load-carrying capacity, i.e., they correspond to tolerance criteria applicable to normal use
shake down	linear elastic structural behaviour is established after yielding of the material has occurred
slamming	impact load on an approximately horizontal member from a rising water surface as a wave passes The direction of the impact load is mainly vertical.
specified minimum yield strength (SMYS)	minimum yield strength prescribed by the specification or standard under which the material is purchased
specified value	minimum or maximum value during the period considered This value may take into account operational requirements, limitations and measures taken such that the required safety level is obtained.
splash zone	external or internal surfaces of a structure which are intermittently wetted by tide or waves or both
stand-still	condition of a wind turbine generator system that is stopped

<i>Term</i>	<i>Explanation</i>
submerged zone	part of the installation which is below the splash zone, including the scour zone and permanently buried parts
substructure	structural component, which forms a part of the support structure for an offshore wind turbine, extending from the foundation to the tower See also Figure 1-1 .
support structure	structure below the yaw system of the rotor-nacelle assembly and includes tower structure, substructure and foundation See also Figure 1-1 .
target safety level	nominal acceptable probability of structural failure
temporary condition	operational condition that may be a design condition, for example the mating, transit or installation phases
tensile strength	minimum stress level where strain hardening is at maximum or at rupture
tidal range	distance between highest and lowest astronomical tide
tide	regular and predictable movements of the sea generated by astronomical forces
tower	structural component, which forms a part of the support structure for a wind turbine, usually extending from somewhere above the ground level or still water level (for an offshore wind turbine) to just below the nacelle of the wind turbine
ultimate limit states (ULS)	correspond to the limit of the load-carrying capacity, i.e., to the maximum load-carrying resistance
upper scour depth	largest scour depth to be applied in the geotechnical and structural analysis
verification	examination to confirm that an activity, a product or a service is in accordance with specified requirements
welding procedure	specified course of action to be followed in making a weld, including reference to materials, welding consumables, preparation, preheating (if necessary), method and control of welding and post-weld heat treatment (if relevant), and necessary equipment to be used
welding procedure qualification record (WPQR)	record comprising a summary of necessary data needed for the issue of a WPS.
welding procedure qualification test (WPQT)	the process of accomplishing welding and testing of a standardised test piece, as indicated in the WPS
welding procedure specification (WPS)	specification which has been qualified to conform with an agreed qualification scheme
wind turbine	system which converts kinetic wind energy into electrical energy Whenever, in this service specification the term is used to describe the wind turbine in general, it describes the rotor-nacelle assembly together with the support structure, as this is the power generating unit.
yawing	rotation of the rotor axis of a wind turbine about a vertical axis

1.6.3 Definition of symbols

1.6.3.1 Latin characters

d	= bolt diameter
f	= frequency
f	= functional relationship
f_{cck}	= characteristic compressive cylinder strength of concrete according to DNVGL-ST-C502
f_{cd}	= design compressive strength of concrete
f_{ck}	= characteristic compressive cylinder strength of concrete according to EN 1992-1-1
$f_{ck,cube75}$	= characteristic compressive cube strength on 75 mm cubes
f_{cn}	= normalized compressive strength of concrete according to DNVGL-ST-C502
f_{ksd}	= design shell buckling strength for the unmodified shell according to DNVGL-RP-C202
f_R	= excitation frequency
f_u	= nominal lowest ultimate tensile strength
f_{ub}	= ultimate tensile strength of bolt
f_{uw}	= lowest ultimate tensile strength
f_y	= specified minimum yield stress
f_0	= natural frequency
g	= acceleration of gravity
h	= height of shear key in grouted connection
h	= water depth
k	= number of stress blocks
k	= radial stiffness parameter
k	= wave number
k_{eff}	= effective spring stiffness for the shear keys
k_{rD}	= support spring stiffness
k_s	= hole clearance factor
l_e	= elastic length of pile
n	= number
p	= pressure, nominal pressure
p_{local}	= local pressure
p_{nom}	= nominal pressure
s	= spacing between shear keys
s_{eff}	= effective vertical distance between shear keys
t_{eff}	= effective thickness
t_g	= thickness of grout

t_{JL}	= thickness of jacket leg
t_p	= wall thickness of pile
t_s	= wall thickness of sleeve
$t_{test,min}$	= temperature which the equipment, constituent materials and test and curing environments shall be maintained at during material testing of grout to be qualified for application at temperatures below +5°C (see also DNVGL-ST-C502)
t_{TP}	= wall thickness of transition piece
t_w	= throat thickness
u	= displacement
w	= width of shear key
A_s	= net area in the threaded part of a bolt
D	= diameter
D_c	= characteristic cumulative damage
D_d	= design cumulative damage
D_{JL}	= diameter of jacket leg
D_p	= diameter of pile
E	= Young's modulus
E_g	= Young's modulus of grout
E_{sk}	= Young's modulus of steel reinforcement
E	= environmental load
F	= force, load
F_d	= design load
F_k	= characteristic load
F_p	= preloading force in bolt
F_{H1Shk}	= tangential (horizontal) force on one vertical shear key
F_{V1Shk}	= vertical force on one shear key
G	= shear modulus
H	= wave height
H_s	= significant wave height
KC	= Keulegan-Carpenter number
L	= nominal span of a beam
L	= total length of grouted connection, i.e. full length from the grout packers/seals to the outlet holes/location of overflow
L_g	= effective length of grouted connection
L_s	= length of vertical shear key
M	= moment
M_p	= plastic moment resistance

M_T	= torque
M_y	= elastic moment resistance
N	= fatigue life, i.e. number of cycles to failure
P	= load
P	= axial force
Q	= variable functional load
R	= radius
R	= resistance
R_d	= design resistance
R_k	= characteristic resistance
R_p	= outer radius of pile
R_s	= outer radius of sleeve
R_{TP}	= outer radius of transition piece
S	= scour depth
S_d	= design load effect
S_k	= characteristic load effect
T	= wave period
T_C	= design useful life of coating
T_D	= design life of structure
V_{corr}	= expected maximum corrosion rate
W	= steel with improved weldability
Z	= steel grade with proven through thickness properties with respect to lamellar tearing

1.6.3.2 Greek characters

α	= angle
α	= cone angle of conical grouted connection, measured from the vertical plane
α	= current amplification factor
β_{cc}	= coefficient for considering the time-dependent strength increase in concrete and grout
β_w	= correlation factor
δ	= deflection
$\Delta\sigma$	= stress range
ε	= strain
φ_0	= lowest eigenvalue of gravity load factor for flexural buckling of columns
γ_f	= load factor

γ_c	= material factor for concrete
γ_m	= material factor
γ_M	= material factor
γ_{Mw}	= material factor for welds
γ_s	= material factor for reinforcement
γ_{Sd}	= partial safety factor to consider the inaccuracies of the model for stress calculation
λ	= slenderness
θ	= rotation angle
μ	= friction coefficient
ν	= Poisson's ratio
ρ	= contact pressure
σ	= stress
σ_d	= design stress
σ_k	= characteristic stress
$\sigma_{x,Rd}$	= design shell buckling strength for the unmodified shell according to EN 1993-1-6
τ	= shear stress
τ_d	= design shear stress

1.6.3.3 Subscripts

c	= characteristic value
d	= design value
k	= characteristic value
p	= plastic
y	= yield

1.6.4 Abbreviations

Table 1-6 Acronyms and abbreviations

<i>Abbreviation</i>	<i>Description</i>
AISI	American Iron and Steel Institute
ALS	accidental limit states
API	American Petroleum Institute
ASTM	American Society for Testing and Materials

<i>Abbreviation</i>	<i>Description</i>
BS	British Standard (issued by British Standard Institute)
BSH	Bundesamt für Seeschifffahrt und Hydrographie
CD	chart datum
CP	cathodic protection
CPT	cone penetration test
CTOD	crack tip opening displacement
CVI	close visual inspection
DFF	design fatigue factor
DFT	dry film thickness
EHS	extra high strength
EXC	execution classe
FLS	fatigue limit states
GACP	galvanic anode cathodic protection
GBS	gravity-based structure
GVI	general visual inspection
HAT	highest astronomical tide
HAZ	heat-affected zone
HISC	hydrogen induced stress cracking
HS	high strength
HSWL	highest still water level
ICCP	impressed current cathodic protection
IEC	International Electrotechnical Commission
ISO	International Organization for Standardization
LAT	lowest astronomical tide
LDD	load duration distribution
LSWL	Lowest still water level
MP	monopile
MWL	mean water level
NACE	National Association of Corrosion Engineers
NDT	non-destructive testing
NS	normal strength
O&M	operations and maintenance
PWHT	post weld heat treatment

<i>Abbreviation</i>	<i>Description</i>
RHS	rectangular hollow section
RNA	rotor-nacelle assembly
ROV	remotely operated vehicle
SCF	stress concentration factor
SLS	serviceability limit states
SMYS	specified minimum yield stress
SWL	still water level
TP	transition piece
ULS	ultimate limit states
WPQR	welding procedure qualification record
WPQT	welding procedure qualification test
WPS	welding procedure specification

1.7 Procedural requirements

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

The provision for using non-DNV GL standards is that the same safety level as the one resulting for designs according to this standard is obtained.

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

When code checks are performed according to other codes than DNV GL standards, the load and resistance factors as given in the respective codes shall be taken as minimum values.

National and governmental regulations may supplement or overrule the requirements of this standard as applicable.

1.7.1 Documentation requirements - equivalence and future developments

This standard specifies requirements for the design of wind turbine support structures intended to ensure a safety level that is deemed acceptable for such structures. Some of these requirements imply certain constraints on structural designs that reflect the current practice in the industry and established principles of design and construction of wind turbine support structures. Alternative designs and arrangements that deviate from these requirements may be accepted provided that it is documented that the level of safety is at least as high as that implied by the requirements of this standard. Technology qualification procedures may be helpful in this context.

Guidance note:

A recommended method for identifying risk control options and documenting the safety of alternative designs and arrangements is given in DNVGL-RP-A203.

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1.7.2 Certification requirements

Certification principles and procedures related to certification services for wind turbine support structures are specified in relevant DNV GL service specifications as according to DNVGL-SE-0073, DNVGL-SE-0074 and DNVGL-SE-0190.

SECTION 2 DESIGN PRINCIPLES

2.1 Introduction

2.1.1 General

2.1.1.1 This section describes design principles and design methods for wind turbine structural design, including:

- design by partial safety factor method
- design assisted by testing
- probability-based design.

2.1.1.2 General design conditions regardless of design method are also given in [2.2].

2.1.1.3 The DNV GL standards for wind turbine design are based on the partial safety factor method, which is based on separate assessment of the load effect in the structure due to each applied load process. The standard also describes design by direct simulation of the combined load effect of simultaneously applied load processes, which is useful in cases where it is not feasible to carry out separate assessments of the different individual process-specific load effects.

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

2.1.1.5 Structural reliability analysis methods for direct probability-based design are mainly considered as applicable to special case design problems, to calibrate the load and material factors to be used in the partial safety factor method, and to design for conditions where limited experience exists.

2.1.2 Aim of the design

Structures and structural elements shall be designed to:

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

2.2 General design conditions

2.2.1 General

2.2.1.1 The design of a structural system, its components and details shall, as far as possible, satisfy the following requirements:

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

2.2.1.2 Structures and structural components shall possess ductile resistance unless the specified purpose requires otherwise.

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

2.3 Safety classes

2.3.1 Safety classes

2.3.1.1 Wind turbine support structures shall be designed according to one of the following two safety classes:

- the normal safety class which applies when a failure results in risk of personal injury and / or economic, environmental or social consequences
- the special safety class which applies when the safety requirements are determined by local regulations and / or the safety requirements are agreed between the designer and the customer.

2.3.1.2 Partial safety factors for the loads acting upon a wind turbine support structure of the normal safety class are specified in DNVGL-ST-0437 and [Sec.3](#) of this standard. Partial safety factors for materials for the normal safety class are given in this standard.

2.3.1.3 Partial safety factors for wind turbine support structures of the special safety class require prior agreement. A wind turbine designed according to the special safety class is a “class S” turbine.

2.3.1.4 The safety class is the same, regardless of which design philosophy is applied regarding inspection level etc.

Guidance note:

A design of a structural component which is based on an assumption of inspections and possible maintenance and repair throughout its design life may benefit from a reduced structural dimension, e.g. a reduced cross-sectional area, compared to that of a design without such an inspection and maintenance plan, in order to achieve the same safety level for the two designs.

This refers in particular to designs which are governed by the FLS or the SLS. It may be difficult to apply this to designs which are governed by the ULS or the ALS.

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2.3.1.5 The target safety level of the normal safety class in this standard is a nominal annual probability of failure of 10^{-4} .

Guidance note:

The target safety level is the safety level aimed at for the entire structure and will in practice also be the safety level for individual failure modes, since one failure mode is usually dominating. It is intended for use both in case of local failures in hot spots and in case of failures with system effects, such as failure in the weakest link of a series system.

Designs are accepted with achieved failure probabilities to either side of the nominal target failure probability of 10^{-4} . The nominal maximum annual acceptable failure probability is $5 \cdot 10^{-4}$. This maximum acceptable failure probability corresponds to what IEC 61400-1 specifies as its target failure probability, but which in background documentation for IEC 61400-1 appears to be a maximum acceptable failure probability.

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2.4 Limit states

2.4.1 General

2.4.1.1 A limit state is a condition beyond which a structure or structural component will no longer satisfy the design requirements.

2.4.1.2 The following limit states are considered in this standard:

- ultimate limit states (ULS) correspond to the maximum load-carrying resistance
- fatigue limit states (FLS) correspond to failure due to the effect of dynamic loading
- accidental limit states (ALS) correspond to (1) maximum load-carrying capacity for (rare) accidental loads or (2) post-accidental integrity for damaged structures
- serviceability limit states (SLS) correspond to tolerance criteria applicable to normal use.

2.4.1.3 Examples of limit states within each category:

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 (which in some cases is reduced due to repetitive loading) or the ultimate deformation of the components
- transformation of the structure into a mechanism (collapse or excessive deformation).

FLS:

- cumulative damage due to repeated loads.

ALS:

- structural damage caused by accidental loads (ALS type 1)
- ultimate resistance of damaged structures (ALS type 2)
- loss of structural integrity after local damage (ALS type 2).

SLS:

- deflections that may alter the effect of the acting forces
- excessive vibrations producing discomfort or affecting non-structural components
- excessive vibrations affecting turbine operation and energy production
- deformations or motions that exceed the limitation of equipment
- durability
- differential settlements of foundations soils causing intolerable tilt of the wind turbine
- temperature-induced deformations.

2.5 Design by the partial safety factor method

2.5.1 General

2.5.1.1 The partial safety factor 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 values of the governing variables and subsequently fulfilling a specified design criterion expressed in terms of these factors and these characteristic values. The governing variables consist of:

- loads acting on the structure or load effects in the structure
- resistance of the structure or strength of the materials in the structure.

2.5.1.2 The characteristic values of loads and resistance, or of load effects and material strengths, are chosen as specific quantiles in their respective probability distributions. The requirements to the load and resistance factors are set such that possible unfavourable realisations of loads and resistance, as well as their possible simultaneous occurrences, are accounted for to an extent which ensures that a satisfactory safety level is achieved.

2.5.2 The partial safety factor format

2.5.2.1 The safety level of a structure or a structural component is considered to be satisfactory when the design load effect S_d does not exceed the design resistance R_d :

$$S_d \leq R_d$$

This is the design criterion. The design criterion is also known as the design inequality. The corresponding equation $S_d = R_d$ forms the design equation.

Guidance note:

The load effect S may be any load effect such as an external or internal force, an internal stress in a cross-section, or a deformation, and the resistance R against S is the corresponding resistance such as a capacity, a yield stress or a critical deformation.

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2.5.2.2 There are two approaches to establish the design load effect S_{di} associated with a particular load F_i :
(1) The design load effect S_{di} is obtained by multiplication of the characteristic load effect S_{ki} by a specified load factor γ_{fi} :

$$S_{di} = \gamma_{fi} S_{ki}$$

where the characteristic load effect S_{ki} is determined in a structural analysis for the characteristic load F_{ki} .

(2) The design load effect S_{di} is obtained from a structural analysis for the design load F_{di} , where the design load F_{di} is obtained by multiplication of the characteristic load F_{ki} by a specified load factor γ_{fi} :

$$F_{di} = \gamma_{fi} F_{ki}$$

Approach (1) shall be used to determine the design load effect when a proper representation of the dynamic response is the prime concern, whereas approach (2) shall be used if a proper representation of nonlinear material behaviour or geometrical nonlinearities or both are the prime concern. Approach (1) typically applies to the determination of design load effects in the support structure from the wind loading on the turbine, whereas approach (2) typically applies to the design of the support structure with the load effects in the tower applied as a boundary condition.

Guidance note:

For structural design of an onshore gravity-based concrete foundation approach (2) can be used to properly account for the influence from the nonlinearities of the soil. In a typical design situation for an onshore gravity-based concrete foundation the main loads are wind loads in addition to permanent loads. The design wind load effects at an appropriate interface level, such as at the tower bottom flange, may be determined from an integrated structural analysis of the support structure by approach (1) and consist of a shear force in combination with a bending moment and a torsion moment. These design load effects may then be applied as external design loads at the interface level, and the design load effects in the gravity-based concrete foundation for these design loads may further be determined from a structural analysis of the gravity-based concrete foundation by approach (2).

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2.5.2.3 The design load effect S_d is the most unfavourable combined load effect resulting from the simultaneous occurrence of n loads F_i , $i = 1, \dots, n$. It may be expressed as:

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

where f denotes a functional relationship.

According to the partial safety factor format, the design combined load effect S_d resulting from the occurrence of n independent loads F_i , $i = 1, \dots, n$, may be taken as:

$$S_d = \sum_{i=1}^n S_{di}(F_{ki})$$

where $S_{di}(F_{ki})$ denotes the design load effect corresponding to the characteristic load F_{ki} .

When there is a linear relationship between the load F_i acting on the structure and its associated load effect S_i in the structure, the design combined load effect S_d resulting from the simultaneous occurrence of n loads F_i , $i = 1, \dots, n$, may be achieved as:

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

Guidance note:

As an example, the combined load effect could be the bending stress in a vertical foundation pile, resulting from a wind load and a wave load that act concurrently on a structure supported by the pile.

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2.5.2.4 When there is a linear relationship between the load F_i and its load effect S_i , the characteristic combined load effect S_k resulting from the simultaneous occurrence of n loads F_i , $i = 1, \dots, n$, may be achieved as:

$$S_k = \sum_{i=1}^n S_{ki}$$

2.5.2.5 Characteristic load effect values S_{ki} are obtained as specific quantiles in the distributions of the respective load effects S_i . In the same manner, characteristic load values F_{ki} are obtained as specific quantiles in the distributions of the respective loads F_i .

Guidance note:

Which quantiles are specified as characteristic values may depend on which limit state is considered. Which quantiles are specified as characteristic values may also vary from one specified combination of load effects to another.

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2.5.2.6 In this standard, design in the ULS shall either be based on a characteristic combined load effect S_k defined as the 98% quantile in the distribution of the annual maximum combined load effect, or on a characteristic load F_k defined as the 98% quantile in the distribution of the annual maximum of the combined load. The result is a combined load or a combined load effect whose return period is 50 years.

Guidance note:

When n load processes occur simultaneously, the standard specifies more than one set of characteristic load effects (S_{k1}, \dots, S_{kn}) to be considered in order for the characteristic combined load effect S_k to come out as close as possible to the 98% quantile. For each specified set (S_{k1}, \dots, S_{kn}), the corresponding design combined load effect is determined according to [2.5.2.3]. For use in design, the design combined load effect S_d is selected as the most unfavourable value among the design combined load effects that result for these specified sets of characteristic load effects.

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2.5.2.7 When the structure is subjected to the simultaneous occurrence of n load processes, and the structural behaviour, e.g. the damping, is influenced by the character of at least one of these loads, then it may not always be feasible to determine the design load effect S_d , resulting from the simultaneous occurrence of the n loads, by a linear combination of separately determined individual load effects as set forth in [2.5.2.3]. Within the framework of the partial safety factor method, the design combined load effect S_d , resulting from the simultaneous occurrence of the n loads, may then be established as a characteristic combined load effect S_k multiplied by a common load factor γ_f . The characteristic combined load effect S_k will in this case need to be defined as a quantile in the upper tail of the distribution of the combined load effect that results in the structure from the simultaneous occurrence of the n loads. In principle, the distribution of this combined load effect comes about from a structural analysis in which the n respective load processes are applied simultaneously.

Guidance note:

The total damping of a wind turbine depends on the wind loading and its direction relative to other loads, such that for example the wave load effect in an offshore support structure becomes dependent on the characteristics of the wind loading. Unless the wind load characteristics can be properly accounted for to produce a correct total damping and a correct separate wave load effect in a structural analysis for the wave load, then the structure may need to be analysed for the sought-after combined load effect for a simultaneous application of the wind load process and the wave load process.

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2.5.2.8 The resistance R against a particular load effect S is, in general, a function of parameters such as geometry, material properties, environment, and load effects themselves, the latter through interaction effects such as degradation.

2.5.2.9 There are two approaches to establish the design resistance R_d of the structure or structural component:

- 1) The design resistance R_d is obtained by dividing the characteristic resistance R_k by a specified material factor γ_m :

$$R_d = \frac{R_k}{\gamma_m}$$

- 2) The design resistance R_d is obtained from the design material strength σ_d by a capacity analysis:

$$R_d = R(\sigma_d)$$

in which R denotes the functional relationship between material strength and resistance and in which the design material strength σ_d is obtained by dividing the characteristic material strength σ_k by a material factor γ_m :

$$\sigma_d = \frac{\sigma_k}{\gamma_m}$$

Which of the two approaches applies depends on the design situation. In this standard, the approach to be applied is specified from case to case.

2.5.2.10 The characteristic resistance R_k is obtained as a specific quantile in the distribution of the resistance. It may be obtained by testing, or it may be calculated from the characteristic values of the parameters that govern the resistance. In the latter case, the functional relationship between the resistance and the governing parameters is applied. Likewise, the characteristic material strength σ_k is obtained as a specific quantile in the probability distribution of the material strength and may be obtained by testing.

2.5.2.11 Load factors account for:

- possible unfavourable deviations of the loads from their characteristic values
- the probability that different loads exceed their respective characteristic values simultaneously
- uncertainties in the model and analysis used for determination of load effects.

2.5.2.12 Material factors account for:

- possible unfavourable deviations in the resistance of materials from the characteristic value
- uncertainties in the model and analysis used for determination of resistance
- a possibly lower characteristic resistance of the materials in the structure, as a whole, as compared with the characteristic values interpreted from test specimens.

2.5.3 Characteristic load effect

2.5.3.1 For operational design conditions, the characteristic value S_k of the load effect resulting from an applied load combination is defined as follows, depending on the limit state:

- For load combinations relevant for design against the ULS, the characteristic value of the resulting load effect is defined as a load effect with an annual probability of exceedance equal to or less than 0.02, i.e. a load effect whose return period is at least 50 years.
- For load combinations relevant for design against the FLS, the characteristic load effect history is defined as the expected load effect history.
- For load combinations relevant for design against the SLS, the characteristic load effect is a specified value, dependent on operational requirements.

Load combinations to arrive at the characteristic value S_k of the resulting load effect are given in DNVGL-ST-0437.

2.5.3.2 For temporary design conditions, the characteristic value S_k of the load effect resulting from an applied load combination is a specified value, which shall be selected dependent on the measures taken to achieve the required safety level. The value shall be specified with due attention to the actual location, the season of the year, the duration of the temporary condition, the weather forecast, and the consequences of failure.

2.5.4 Characteristic resistance

2.5.4.1 The characteristic resistance is defined as the 5% quantile in the distribution of the resistance, unless otherwise stated.

2.5.5 Load combinations and resistance factors

2.5.5.1 Load combinations factors for wind turbine loads and permanent loads are given in DNVGL-ST-0437 and [Sec.3](#) of this standard. Resistance factors for the various limit states are given in the respective design section of this standard.

2.5.6 Design by direct simulation of combined load effect of simultaneous load processes

2.5.6.1 Design by direct simulation of the combined load effect of simultaneously acting load processes is an example of approach (1) described in [\[2.5.2.2\]](#). It is based on a direct simulation of the characteristic combined load effect from the simultaneously applied load processes instead of being based on a linear

combination of individual characteristic load effects determined separately for each of the applied load processes.

2.5.6.2 For design of wind turbine structures which are subjected to two or more simultaneously acting load processes, design by direct simulation of the combined load effect is a more attractive approach than design by the linear load combination model of the partial safety factor method. The linear combination model of the partial safety factor method may be inadequate in cases where the load effect associated with one of the applied load processes depends on structural properties which are sensitive to the characteristics of one or more of the other load processes.

Guidance note:

The aerodynamic damping of a wind turbine depends on whether there is wind or not, whether the turbine is in power production or at stand-still, and whether the wind is aligned or misaligned with other loads such as wave loads for offshore structures. Unless correct assumptions can be made about the aerodynamic damping of the wind turbine in accordance with the actual status of the wind loading regime, separate determination of the load effect due to wave load alone to be used with the partial safety factor format may not be feasible.

In a structural time domain analysis of the turbine subjected concurrently to both wind and wave loading, the aerodynamic damping of the turbine will come out right since the wind loading is included, and the resulting combined load effect, usually obtained by simulations in the time domain, form the basis for interpretation of the characteristic combined load effect.

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2.5.6.3 For design of wind turbine structures which are subjected to two or more simultaneously acting load processes, the design inequality:

$$S_d \leq R_d$$

applies. The design combined load effect S_d is obtained by multiplication of the characteristic combined load effect S_k by a specified load factor γ_f :

$$S_d = \gamma_f S_k$$

2.5.6.4 The characteristic combined load effect S_k may be established directly from the distribution of the annual maximum combined load effect that results from a structural analysis, which is based on simultaneous application of the two or more load processes. In the case of ULS design, the characteristic combined load effect S_k shall be taken as the 98% quantile in the distribution of the annual maximum combined load effect, i.e. the combined load effect whose return period is 50 years.

Guidance note:

There may be several ways in which the 98% quantile in the distribution of the annual maximum combined load effect can be determined. Regardless of the approach, a global structural analysis model should be established, e.g. in the form of a beam-element based frame model, to which loads from several simultaneously acting load processes may be applied.

A structural analysis in the time domain is usually carried out for a specified environmental state of duration typically 10 minutes or one or 3 hours, during which period of time stationary conditions are assumed with constant intensities of the involved load processes. The input consists of concurrent time series of the respective load processes, e.g. wind load and wave load, with specified directions. The output consists of time series of load effects in specified points in the structure.

In principle, determination of the 98% quantile in the distribution of the annual maximum load effect requires structural analyses to be carried out for a large number of environmental states, viz. all those states that contribute to the upper tail of the distribution of the annual maximum load effect. Once the upper tail of this distribution has been determined by integration over the results for the various environmental states, weighted according to their frequencies of occurrence, the 98% quantile in the distribution can be interpreted.

The computational efforts may be considerably reduced when it can be assumed that the 98% quantile in the distribution of the annual maximum load effect can be estimated by the expected value of the maximum load effect in the environmental state whose return period is 50 years.

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2.5.6.5 The characteristic resistance shall be calculated as described in [2.5.4].

2.6 Design assisted by testing

2.6.1 General

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

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

2.6.1.3 To the extent that testing is used for design, the testing shall be verifiable.

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

Full-scale tests or monitoring of 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.


2.7 Probability-based design

2.7.1 Definition

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

2.7.2 General

2.7.2.1 This subsection gives requirements for structural reliability analysis undertaken in order to document compliance with this standards.



2.7.2.2 Acceptable procedures for structural reliability analyses are documented in DNV Classification Notes No. 30.6.

2.7.2.3 Reliability analyses shall be based on level 3 reliability methods. These methods utilise probability of failure as a measure of safety and require knowledge of the probability distribution of all governing load and resistance variables.

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

- calibration of a 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 partial safety factor method applied in this standard)
- special case design problems
- novel designs for which limited or no experience exists.

2.7.2.5 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.

2.7.2.6 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, shall 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.

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

- transferable target reliabilities for similar existing design solutions
- internationally recognised codes and standards
- DNV Classification Notes No. 30.6.

SECTION 3 SITE CONDITIONS AND LOADS

3.1 Introduction

3.1.1 General

3.1.1.1 The requirements in this section define and specify site conditions and loads to be considered for design of wind turbine support structures. The load specification includes requirements for calculating load effects and for load combinations.

3.1.1.2 Site conditions shall in general be determined according to the requirements in DNVGL-ST-0437. Additional site conditions to consider for support structure design is specified in [3.2].

3.1.1.3 The environmental loads shall in general be determined according to DNVGL-ST-0437. Additional requirements for determine these load and load effects are given in [3.4].

3.1.1.4 Use of alternative standards for site condition and load calculation together with this standard shall be carried out according to the requirements in [1.2.4].

3.2 Site conditions

3.2.1 General

3.2.1.1 Requirements for soil investigations and geotechnical data are specified in [7.3] of this standard.

3.2.1.2 Site conditions needed for analysis of the environmental loads on the wind turbine shall be determined as specified in DNVGL-ST-0437. Examples for relevant site conditions are; wind climate, air density, temperature, snow and ice, together with; water level, seabed level, wave climate, current and marine growth for offshore wind turbines.

3.2.1.3 Other site conditions like seismicity and the risk of ship collision for offshore structures shall also be considered as specified in DNVGL-ST-0437.

3.2.2 Additional site conditions for offshore wind turbines

3.2.2.1 The salinity of the seawater shall be addressed as a parameter of importance for the design of cathodic protection (CP) systems.

Guidance note:

In estuary and other near-coastal areas the salinity may be affected by river flows and there may be industrial pollutants present in the water which could affect the CP system design.

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3.2.2.2 The presence of pipelines, cables, wrecks or other obstacles within the area of installation of the offshore wind turbines shall be mapped.

3.3 Permanent loads

3.3.1 General

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

- mass of structure
- mass of permanent ballast and equipment
- pre-tension loads
- external and internal hydrostatic pressure of a permanent nature
- reaction to the above, e.g. articulated tower base reaction.

3.3.1.2 The characteristic value 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.

3.4 Calculation of environmental loads

3.4.1 General

3.4.1.1 The environmental loads acting on the wind turbine support structure shall be determined according to DNVGL-ST-0437.

3.4.1.2 For calculating the wind turbine support structure loads the knowledge about the support structure response is essential. Some of the relevant issues to consider are stiffness and damping of the structure. It might therefore be necessary to determine the loads and structural layout in an iterative way.

3.4.1.3 For the calculation of loads for concrete structures and hybrid steel/concrete structures the load-dependent stiffness reduction shall be considered as described in [5.8.3] of this standard.

3.4.1.4 For calculation of the response from pile supported foundations it might be relevant to consider the requirements for calculating the soil response by non-linear elastic elements.

3.4.1.5 For design of offshore wind turbines the possible seabed levels (upper and lower levels) shall be considered.

3.4.1.6 For offshore wind turbines the effect of possible scour shall also be considered. Scour is the result of erosion of soil particles at and nearby a foundation and is caused by waves and current. Scour may have impact on the stiffness and the geotechnical capacity of a foundation and thereby on the structural response that governs the ultimate and fatigue load effects in structural components. Requirements for scour prevention are given in [Sec.8](#).

3.4.1.7 For offshore support structures also the impact from wave run-up shall be considered. See also [3.8].

3.5 Deformation loads

3.5.1 General

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

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

3.5.2 Temperature loads

3.5.2.1 Structures shall be designed for the most extreme temperature differences they may be exposed to. The temperature range for normal environmental conditions shall be taken as specified in DNVGL-ST-0437.

3.5.2.2 Extreme values of high and low temperatures shall be expressed in terms of the most probable highest and lowest values, respectively, with their corresponding return periods.

3.5.2.3 The ambient sea or air temperature shall be calculated as the extreme value whose return period is 50 years.

3.5.2.4 Structures in air shall be designed for a solar radiation intensity of 1000 W/m^2 .

3.5.2.5 For offshore structures both air and seawater temperatures shall be considered when describing the temperature environment.

3.5.2.6 Temperature loads shall be considered as environmental loads.

3.5.3 Geometrical tolerances and settlements

3.5.3.1 All influences on the structure's response from geometrical tolerances in the construction and from settlements of the soil shall be considered in the design and analysis of the support structure. An example of such effects is the one associated with a tilt of the axis of a tower and monopile installation as described in [3.10.2.2].

3.5.4 Settlements

3.5.4.1 Settlement of the support structure and its foundation due to vertical deformations of the supporting soil layers shall be considered. This includes consideration of differential settlements.

3.6 Resonance assessment

3.6.1 Resonance phenomena

3.6.1.1 Vibrations in primary structures induced by resonance phenomena shall be avoided or the effect shall be included in the design.

Guidance note:

Resonance phenomena may implicate deformations and stresses exceeding the structural capacity and may lead to a total damage of the structure.

Even if not doing so, resonance phenomena are most likely to implicate a considerable amount of fatigue being quite rapidly accumulated in members and joints.

A dynamic investigation should be performed to avoid the risk of resonance between structural eigen-modes and energy-rich dynamic loads being excited.

Variation of significant input values that influence the overall stiffness should be considered within a sensitivity study, in particular soil stiffness, scour depth and corrosion allowance for offshore structures.

For the wind turbine support structure design to prevent lock-in to rotor induced vibrations [3.6.2] specifies limits to the natural frequencies of the integral tower and foundation structure.

As for requirements to avoid lock-in to frequencies of vortices shed from a structural element (due to wind, current or wave considering both vibrations in line with or transverse to the action causing the vortex shedding) [3.6.3] gives detailed guidance.

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

3.6.1.2 Vibrations in secondary structures such as internal and external J-tubes, which are induced by resonance phenomena, are undesirable.

3.6.1.3 An assessment of vibrations in J-tubes and other slender elements shall be performed, either based on experience from similar structures or by calculations.

Guidance note:

Vibrations in J-tubes need to be properly considered. For designs involving free-hanging cables, long-term impacts from vibrations should be considered.

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

3.6.2 Vibrations of wind turbine including support structure

3.6.2.1 Present section regards vibrations of the tower and foundation considered at all stages prior to commissioning as well as a completely assembled structure including the RNA.

3.6.2.2 Rotor-induced vibrations

The ratio of the natural frequencies f_0 of the complete system to the excitation frequencies f_R of the turning rotor shall be determined. Excitation frequencies are in particular the rotor speed and the blade frequency.

In general, the following condition shall be fulfilled:

$$\frac{f_R}{f_{0,n}} \leq 0.95 \quad \text{or} \quad \frac{f_R}{f_{0,n}} \geq 1.05$$

where:

f_R designates the excitation frequency, in particular the rotating frequency range of the rotor in the normal operating range and transition frequency of the rotor blades, and

$f_{0,n}$ is the n^{th} natural frequency of the tower and the foundation respectively complete system.

This requirement may be omitted if countermeasures are in place to prevent resonance effects, in particular an operational vibration monitoring system or damping devices.

3.6.2.3 The number of natural frequencies to be determined, n , shall be selected large enough so that the highest calculated natural frequency lies at least 20% higher than the blade transition frequency.

3.6.2.4 The natural frequencies of the compound tower and foundation including RNA shall be determined and specified for the vibration system to be investigated, assuming elastic behaviour of the material. Soil parameters considering the action of cyclic loads shall be used for the ground.

Guidance note:

Particularly in the case of pile foundations, the rotation about the vertical axis and the horizontal displacement of the foundation should be considered in addition to the rotation about the horizontal axes.

It is recommended to design the turbine foundation equipped with a system for monitoring structural vibrations in such cases where the ratio of $f_R/f_{0,n}$ deviate from the interval limits specified above with less than 5%. The monitoring system should be capable of unmistakably detecting a resonance building up. As part of this monitoring design, the operational manual for the structure (pertaining to all stages of the structure's life) should specify the actions to be taken swiftly in any case of such resonance phenomena occurring.

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

3.6.2.5 In order to take account of the uncertainties in calculating the natural frequencies, they shall be varied by $\pm 5\%$.

Guidance note:

The 5% margin is an estimate based on experience not meant to cover soil variation, but other effects like mass and stiffness variation, use of different software and others.

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

3.6.2.6 In particular, monopile foundations and tubular towers shall be designed so that lock-in to the frequencies of vortex shedding will not occur in any stage of the structures' life.

Guidance note:

More specifically, this requirement also regards all stages prior to the commissioning of the turbine (i.e. storage, transportation, installation) where the structures' sensitivity to vortex-induced vibrations may be higher than in the permanent installation because of the temporarily increased ratio of stiffness to inertia.

Vortex-induced vibrations may be prevented by designing for an arrangement of temporary wind spoilers and/or by designing for arranging the structural components and their temporary supports appropriately.

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

3.6.3 Vibrations of structural members and equipment

3.6.3.1 Present section regards vibrations of structural members and equipment. The requirements and advice stated pertain to both primary and secondary parts of the wind turbine installation.

3.6.3.2 The structural members as well as the equipment shall be designed so that cases of their responses becoming resonant with dynamic actions can be considered avoided.

Guidance note:

Dynamic responses should be considered regardless of them being caused by direct actions – such as wind, wave or current – or associated with slave-to-master effects (as can be the case for internal free hanging cables). The design of stiff members may most appropriately include variations of sections and/or arrangements of supports.

For flexible equipment such as free hanging cables preventions of resonance can be achieved through the design of an appropriate number of intermediate lateral supports.

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

3.6.3.3 The structural members shall be designed so that lock-in to the frequencies of vortex shedding will not occur in any stage of the structure's life.

Guidance note:

The particle velocities assumed in analyses of vortex induced vibrations (VIV) should be characteristic for the location of the member considered and for the direction of action relative to the member.

Methods for prediction of lock-in to VIV are given in DNVGL-RP-C205 Sec.9 for actions of wind, wave and current and for oscillations being in line with or transverse to the action.

For onshore towers, methods according to EN 1991-1-4 may be applied.

Structural members, which do not lock in to the frequency of vortex shedding, need not be individually examined for VIV-induced accumulation of fatigue.

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

3.7 Variable functional loads

3.7.1 General

3.7.1.1 Variable functional loads are loads which may vary in magnitude, position and direction during the period under consideration, and which are related to operations and normal use of the installation other than power production. Examples are:

- personnel
- crane operational loads
- loads associated with installation operations
- loads from variable ballast and equipment
- stored materials, equipment, gas, fluids and fluid pressure
- boat impacts and loads from fendering for offshore structures.

3.7.1.2 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.

3.7.1.3 Variable loads can contribute to fatigue. In this case characteristic load histories shall be developed based on specified conditions for operation.

Guidance note:

For a specified condition for operation, the characteristic load history is often taken as the expected load history.

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

3.7.1.4 Loads on access platforms and internal structures are used only for local design of these structures and do therefore usually not appear in any load combination for design of primary support structures including foundations.

3.7.1.5 Loads and dynamic factors from maintenance and service cranes on structures are also to be considered.

3.7.1.6 Boat impact loads on offshore structures as specified in DNVGL-ST-0437 are applied in the design of primary support structures and in the design of some secondary structures. Primary support structures exposed to boat impacts shall not suffer such damage that their capacities to withstand the other loads they are going to be exposed to become compromised.

3.7.1.7 In the normal load case DLC 8.5 according to DNVGL-ST-0437 secondary structures such as fenders, boat landings and ladders, shall not suffer damage to such an extent that they lose their respective functions as access structures. See also DNVGL-ST-0437 [4.2.10] and [4.5.8].

3.7.1.8 In the abnormal load case DLC 8.6 according to DNVGL-ST-0437 (i.e. drifting service vessel) secondary structural parts are allowed to become torn off, e.g. by including weak points or by local

strengthening of supporting structural parts, thereby to avoid excessive damage to the primary structure. If the primary structure is designed to be damaged in DLC 8.6 it shall be shown that the damaged structure is able to withstand DLC 8.2 in DNVGL-ST-0437 to allow that repair work may be performed.

3.7.2 Variable functional loads on platform areas

Variable functional loads on platform areas of the support structure shall be based on Table 3-1 unless specified otherwise in the design basis. For offshore wind turbine structures, the platform area of most interest is the external platform, which shall be designed for ice loads, wave loads and boat impacts. The external platform area consists of lay down area and other platform areas. The intensity of the distributed loads depends on local or global aspects as given in Table 3-1.

3.7.2.1 The following notions are used:

- Local design: For example design of plates, stiffeners, beams and brackets
- Primary design: For example design of girders and columns
- Global design: For example design of support structure

Table 3-1 Variable functional loads on platform areas

	Local design		Primary design	Global design (See notes below)
	Distributed load, p (kN/m ²)	Point load, P (kN)	Apply factor to distributed load	Apply factor to distributed load
Storage areas	q	$1.5 q$	1.0	1.0
Lay down areas	q	$1.5 q$	f	f
Area between equipment	5.0	5.0	f	may be ignored
Walkways, staircases and external platforms	4.0	4.0	f	may be ignored
Walkways and staircases for inspection only	3.0	3.0	f	may be ignored
Internal platforms, e.g. in towers	3.0	1.5	f	may be ignored
Areas not exposed to other functional loads	2.5	2.5	1.0	–

Notes:

Point loads shall be applied on an area 100 mm × 100 mm, and at the most severe position, but not added to wheel loads or distributed loads.

For internal platforms, point loads shall be applied on an area 200 mm × 200 mm

q to be evaluated for each case. Lay down areas should not be designed for less than 15 kN/m².

For calculation of P the value of q in kN/m² shall be used.

$f = \min\{1.0 ; (0.5 + 3/\sqrt{A})\}$, where A is the loaded area in m².

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.

3.7.3 Loads on railings

3.7.3.1 Railing shall be designed for a concentrated load of 1.0 kN acting in all relevant direction. Railing shall also be designed for horizontal line load equal to 0.3 kN/m, applied to the top of the railing. The concentrated load and the line load need not be applied simultaneously.

3.7.3.2 Requirements given in EN 50308 should be met when railing, ladders and other structures for use by personnel are designed.

3.8 Air gap for offshore structures

3.8.1 General

3.8.1.1 For determination of the deck elevation of access platforms for offshore support structures which are not designed to resist direct wave forces, a sufficient air gap based on design water level and design wave crest height shall be ensured, such that extreme wave crests up to the height of the design wave crest are allowed to pass without risk of touching the platform. This requirement applies also to any other deck structure which is not designed to resist wave forces.

3.8.1.2 The air gap shall be at least 20% of the 50-year significant wave height, H_s , but with a minimum value of 1.0 m. Installation tolerances, global water level rise (due to global warming) and extreme water level have to be included in the calculation of the total extreme sea elevation in addition to the design wave crest.

Guidance note:

Sufficient air gap is necessary in order to avoid slamming forces on an access platform. The requirements for the air gap are partly intended to account for possible local wave effects due to local seabed topography and shoreline orientation. For large-volume structures, air gap calculation should include a wave diffraction analysis.

The design water level is the high water level with a return period of 50 years. The design wave crest height is the crest height with a return period of 50 years.

For air gap estimation combined probability for water level and wave heights may be considered.

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3.8.1.3 Wave run-up, i.e. water pressed upwards along the surface of the structure or the structural members that support the access platform, shall be considered if relevant, either by including such run-up in the calculation of the necessary air gap or by designing the platform for the loads from such run-up.

3.9 Accidental loads

3.9.1 General

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

- earthquake
- dropped objects
- collision impact
- explosions
- fire
- change of intended pressure difference
- accidental impact from helicopter or other objects.

3.9.1.2 Relevant accidental loads should be determined on the basis of an assessment and relevant experiences.

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

3.9.1.4 It shall be noted that for wind turbine support structures the impact from a drifting services vessel is considered an abnormal load as described in DNVGL-ST-0437 [4.2.10] and [4.5.8]. Drifting service vessel impact is therefore handled as ULS.

3.10 Serviceability loads and requirements

3.10.1 Serviceability loads

3.10.1.1 For serviceability limit states design the following three environmental load cases can be needed:

- characteristic extreme load
- LDD 10^{-4} (i.e. the load level only exceeded 0.01% of the time equivalent to 17.5 h in 20 years)
- LDD 10^{-2} (i.e. the load level only exceeded 1% of the time equivalent to 1750 h in 20 years).

These load cases shall be determined as specified in DNVGL-ST-0437.

3.10.2 Criteria for deflections and geometry of the unloaded structure

3.10.2.1 Limiting values for vertical deflections are usually specified in the design basis. In lieu of such deflection criteria the limits stated in [Table 3-2](#) may be applied.

Table 3-2 Deflection criteria, vertical deflections

<i>Structural component</i>	<i>Limit for δ_{max}</i>	<i>Limit for δ_2</i>
Deck beams	L/200	L/300
<p>L designates the nominal span of the beam. For cantilever beams, L shall be taken as twice the projecting length of the cantilever,</p> <p>δ_{max} designates the resulting sagging of the member relative to the straight line joining the supports: $\delta_{max} = \delta_1 + \delta_2 - \delta_0$</p> <p>$\delta_0$ is the pre-camber,</p> <p>δ_1 is the deflection of the beam due to permanent loads immediately after their application, and</p> <p>δ_2 is the sum of the deflection of the beam due to variable loading and time-dependent deflections due to the permanent load.</p>		

See also [Figure 3-1](#), which illustrates the case of a single-span simply supported beam.

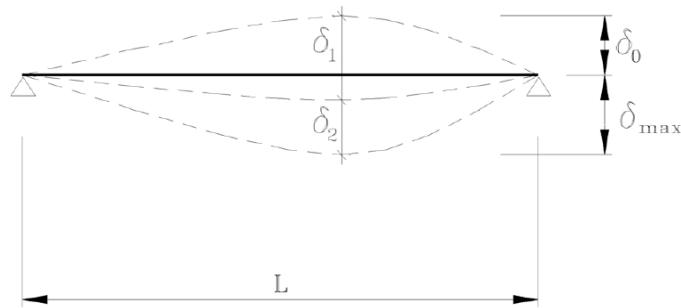


Figure 3-1 Definitions of vertical deflections

3.10.2.2 Maximum acceptable permanent deviations from the ideal vertical configuration of the axis of the unloaded support structure shall be specified in the design basis, including those that develop during the operational phase.

Guidance note:

Realistically, it may be required that the foundation/substructure is installed and the tower is constructed with a total tolerance for the tower axis tilt of 0.25° .

Then, when limiting the total and permanent tilt rotation to say 0.50° , this allows for permanent deformations in the soil to develop and implicate a nominal additional tilt rotation of the tower axis of 0.25° . Plastic soil deformations are typically associated with settlement gradients (for example between individually arranged direct foundations of the feet of a lattice tower) and/or ultimate loads on the support structure (for example in the p-y response of a monopile).

In particular, for an offshore monopile the rotation due to plastic soil deformations is most appropriately defined as the inclination of the line, which is defined by the point of pile rotation at the mud line. See also [Figure 3-2](#).

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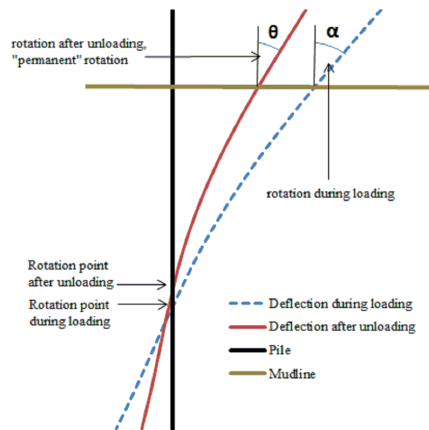


Figure 3-2 Monopile in as-installed and deformed states

3.11 Load effect analysis

3.11.1 General

3.11.1.1 Load effects, in terms of motions, displacements, and internal forces and stresses in the wind turbine support structure, shall be determined with due regard for:

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

3.11.1.2 Permanent loads, functional loads, deformation loads, and fire loads can generally be treated by static methods of analysis. Environmental loads (by wind, waves, current, ice and earthquake) and certain accidental loads (by impacts and 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.

3.11.1.3 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.

3.11.1.4 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.

3.11.1.5 In the final design stage theoretical methods for prediction of important responses of any novel system should be verified by appropriate model tests. Full scale tests may also be appropriate, in particular for large wind farms.

3.11.2 Global motion analysis

3.11.2.1 The purpose of a motion analysis is to determine displacements, accelerations, velocities and hydrodynamic pressures relevant for the loading on the wind turbine support structure. Excitation by waves, current and wind should be considered.

3.11.3 Load effects in structures and foundation soils

3.11.3.1 Displacements, forces and 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.

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

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

3.12 Load combinations for in-place structures

3.12.1 General

3.12.1.1 Environmental load shall be combined as specified in DNVGL-ST-0437 if not specified otherwise in this standard in the subsection in question.

3.12.1.2 Environmental and other variable loads shall be combined with deadloads, temperature actions etc. as appropriate for the structure in question and as specified in the relevant subsections of this standard.

3.12.1.3 Deformation loads shall be included in any load combination with a partial safety factor of $\gamma_f = 1.0$.

3.12.1.4 For variable functional loads as specified in [3.7] the load combination factors as according to Table 3-3 shall apply.

Table 3-3 Load factors γ_f for variable functional loads on platform areas etc.

Variable functional loads		Environmental loads		Permanent loads*		
ULS	FLS, ALS, SLS	ULS	FLS, ALS, SLS	ULS		FLS, ALS, SLS
				Favourable	Unfavourable	
1.25**	1.0	0.7***	1.0	0.9	1.25	1.0

* Permanent loads included for example dead loads and pre-tension loads.
 ** For functional loads from drifting services vessel as specified in [3.7.1.8] the load factor can be taken as 1.1.
 *** When functional loads from boat impacts shall be combined with environmental loads, the environmental load factor shall be increased from 0.7 to 1.0 to reflect that boat impacts are correlated with the wave conditions.

3.13 Transportation and installation loads

Transportation and installation of onshore and offshore wind turbine structures should be planned and executed according to DNVGL-ST-0054 and DNVGL-ST-N001.

SECTION 4 STEEL STRUCTURES

4.1 Steel structure concepts

In this section typical steel support structures are described. For geotechnical elements of the structures such as piles or suction buckets see [Sec.7](#) of this standard. See also [Figure 1-1](#) for further guidance.

4.1.1 Tubular towers

A common tower design for both onshore and offshore wind turbines are tubular steel towers, which are manufactured in tubular sections typically of 20-30 m length with flanges at both ends. The tower will typically have circular cross-sections.

4.1.2 Segmented towers

In a segmented steel tower the cross-sections are divided into a number of steel panels which typically are assembled by bolts. A major advantage for a segmented tower design is the facilitation of transportation.

4.1.3 Lattice towers

Lattice towers are typically manufactured by means of welded or bolted tubular steel profiles or L-section steel profiles. The lattice towers are typically three- or four-legged and consist of corner chords interconnected with bracings in a triangulated structure.

4.1.4 Offshore monopile substructure/foundations

4.1.4.1 The monopile structure is a simple design by which the tower is supported by one large pile, either directly or through a transition piece, which is a transitional section between the tower and the monopile. The monopile continues down into the seabed to a depth where it is fully anchored. The monopile structure is typically made of circular steel tubes and fabricated in one piece.

4.1.4.2 If a transition piece is used this is typically equipped with appurtenances and the transition piece is typically installed on the monopile after the pile has been installed. The transition piece is also typically made of circular steel tubes and fabricated in one piece.

4.1.5 Offshore jacket substructure/foundations

Jackets substructure/foundations are typically three- or four-legged triangulated structures all made of circular steel tubes. On top of the jacket structure is installed a transition piece, typically a plated structure, which is designed with a large centre steel tube for connection with the tower. The jacket is typically anchored into the seabed by piles installed at each jacket leg.

4.1.6 Offshore tripod substructure/foundations

The tripod substructure/foundations is a standard three-legged structure made of circular steel tubes. The central steel shaft of the tripod makes the transition to the wind turbine tower. The tripod can have either vertical or inclined pile sleeves.

4.1.7 Offshore jacket and tripods with suction buckets

The jacket/tripod substructure/foundations with suction buckets are structures equipped with suction bucket foundations at each leg instead of piles as for the conventional jacket/tripod structure. The use of the suction

buckets eliminates the need for driving of piles as required for the conventional jacket/tripod substructure/foundations.

4.1.8 Offshore suction mono-bucket foundations

The suction mono-bucket steel structure typically consists of a centre column connected to a single large steel bucket through flange-reinforced shear panels, which distribute the loads from the centre column to the edge of a large bucket. The wind turbine tower is connected to the substructure centre column above mean sea level. The bucket is installed by means of suction and will in the permanent case behave as a gravity-based foundation, relying on the weight of the soil encompassed by the steel bucket with a skirt length of approximately the same dimension as the width of the bucket.

4.2 Selection of steel materials and structural categories

4.2.1 General

This subsection describes the structural categorisation and selection of steel materials to be applied for on- and offshore steel structures.

4.2.2 Temperatures for steel selection

4.2.2.1 The service temperature is a reference temperature used as a criterion for the selection of steel grades according to DNVGL-OS-B101.

Guidance note:

For steel grade selection according to EN 1993-1-10, as specified in [4.2.4.8], the relevant air temperature to consider is specified in EN 1993-1-10 Sec.2.2.

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

4.2.2.3 External onshore structures shall be designed with service temperatures equal to the design temperature at the area(s) where the unit is to operate. The design temperature is defined as the lowest mean daily temperature in air.

4.2.2.4 External offshore structures above the lowest astronomical tide (LAT) shall be designed for service temperatures equal to the design temperature at the area(s) where the unit is to operate.

4.2.2.5 Materials in offshore 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 the lowest ambient temperature applicable for the relevant water depths.

4.2.3 Structural category

4.2.3.1 The purpose of the structural categorisation is to assure adequate material quality and suitable inspection to avoid brittle fracture. The purpose of inspection is among other things to detect defects that may grow into fatigue cracks during service life.

4.2.3.2 Conditions that may result in brittle fracture shall be avoided.

Guidance note:

Brittle fracture may occur under a combination of:

- presence of sharp defects such as cracks,
- accumulation of hydrogen ions,
- high tensile stress in direction normal to planar defect(s), and
- 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.

The risk of hydrogen embrittlement developing during fabrication is best minimized through a strict control of filler materials storage and an equally strict control of temperature and humidity during welding.

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 geometrically complex connection may show a more complex three-dimensional stress state due to external loading than simple connections. Stresses acting in the through-thickness direction of plates are another item of concern. These stress states may provide basis for a cleavage fracture.

The fracture toughness is dependent on temperature and material thickness. In general, lower temperatures and thicker materials demand for higher toughness. 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 specified. In special cases post weld heat treatment may be performed to reduce crack driving stresses, see also DNVGL-OS-C401 Ch.2 Sec.6. A suitable amount of inspection should be carried out to remove planar defects larger than those considered acceptable. In this standard for wind turbine support structures the 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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4.2.3.3 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 which in combination 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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4.2.3.4 The structural category for selection of materials shall be determined according to principles given in [Table 4-1](#).

Table 4-1 Structural categories for selection of materials

<i>Structural category</i>	<i>Principles for determination of structural category</i>
Special	Structural parts where failure will have substantial consequences and are subject to a complex stress condition. Structural parts with conditions that may increase the probability of a brittle fracture. ¹⁾
Primary	Structural parts where failure will have substantial consequences.
Secondary	Structural parts where failure will be without significant consequence.
Note:	
1) In complex joints a tri-axial 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.	

Guidance note:

Tubular joints are categorised as special due to their biaxial or tri-axial stress patterns and risk of brittle fracture. This will influence the thickness limitations as specified in Table 4-4.

Likewise, flanged connections in primary structures are in general categorized as special due to their irregular stress pattern and loading in through-thickness direction.

Transition pieces in offshore jacket foundations, which are typically designed as plated structures as described in [4.1.5], are in general special structures due to both geometry and stress conditions being quite complex in the geometric adaption as well as in the transfer of loads from tower to jacket.

Tower structures are normally categorised as primary, because they are non-redundant structures whose stress pattern is primarily uniaxial and whose risk of brittle fracture is negligible.

Likewise, monopile structures including the uniaxially-loaded parts of their transition pieces are also categorised as primary.

Ladders, platforms, railings, boat landings and J-tubes are normally categorized as secondary.

If the structure is designed and analysed according to the Eurocode 3 series, tubular joints are categorised as EXC4 due to their biaxial or tri-axial stress patterns and risk of brittle fracture. This will influence the thickness limitations.

Tower structures are normally categorised as EXC3, because they are non-redundant structures whose stress pattern is primarily uniaxial and whose risk of brittle fracture is negligible. Likewise, monopile structures are also categorised as EXC3.

Ladders, platforms, railings, boat landings and J-tubes are normally categorized as EXC2.

Execution classes (EXC) are defined according to EN 1990 and EN 1090-2.

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4.2.4 Structural steel

4.2.4.1 Wherever the subsequent requirements for steel grades are dependent on plate thickness, these requirements are based on the nominal thickness of the base product.

4.2.4.2 The requirements in this subsection deal with the selection of various structural steel grades in compliance with the requirements given in DNVGL-OS-B101.

4.2.4.3 If the structure is designed and analysed according to Eurocode the selection of steel grades shall be in compliance with the requirements stated in EN 10025 series or EN 10225. Where other codes or standards have been specified in the design basis and utilised in the specification of steels, the application of such steel grades within the structure shall be specially considered.

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

4.2.4.5 The material toughness may also be evaluated by fracture mechanics testing (CTOD test) in special cases.

4.2.4.6 In structural cross-joints or other structural elements where high tensile stresses are acting perpendicular to the plane of a rolled plate, the plate material shall be tested to prove the ability to resist lamellar tearing, Z-quality. The required Z-quality shall be based on the ULS design stress perpendicular to the rolling plane.

Guidance note:

For steel structures designed and analysed according to the Eurocode 3 series, the procedure described in EN 1993-1-10 may be applied.

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4.2.4.7 Requirements for forgings and castings are given in DNVGL-OS-B101 or EN 10228-3.

4.2.4.8 Material designations for steel are given in terms of a strength group and a specified minimum yield stress according to steel grade definitions given in DNVGL-OS-B101 Ch.2 Sec.2. Structural steel designations for various strength groups are referred to as given in [Table 4-2](#). For onshore tubular towers that are designed and analysed according to Eurocode the steel grade selection may be carried out according to EN1993-1-10 Chapter 2, Table 2.1.

Table 4-2 Steel grade conversions ¹⁾

	<i>EN 10025-2</i>	<i>EN 10025-3</i>	<i>EN 10025-4</i>	<i>VL grade</i>	<i>Test temperature VL grade (°C)</i>
Normal strength steel (NS)	S235JR	-	-	VL A	-
	S235J0	-	-	VL B	0
	S235J2+N	-	-	VL D	-20
	-	-	-	VL E	-40
High strength steel (HS)	S275J0	S275N	S275M	VL A27S	0
	S275J2+N	S275N	S275M	VL D27S	-20
	-	S275NL	S275ML	VL E27S	-40
	-	-	-	VL F27S	-60
	(S355J0)	S355N	S355M	VL A36	0
	S355K2+N, (S355J2+N)	S355N	S355M	VL D36	-20
	-	(S355NL)	S355ML	VL E36	-40
	-	-	-	VL F36	-60
Extra high strength steel (EHS)	-	S420N	S420M	VL A420	0
	-	S420NL	S420ML	VL D420	-20
	-	(S420NL)	(S420ML)	VL E420	-40
	-	-	-	VL F420	-60
	-	S460NL	S460ML, (S460M)	VL A460	0
	-	(S460NL)	S460ML	VL D460	-20
	-	(S460NL)	(S460ML)	VL E460	-40
	-	-	-	VL F460	-60

Note:

¹⁾ Grades in parentheses compare reasonably well with corresponding VL grades with respect to Charpy V-notch impact requirements.

Guidance note:

Important notes to the conversions between EN grades and VL grades in [Table 4-2](#):

The conversions are based on comparable requirements for strength and toughness.

VL grades are, in general, better steel qualities than comparable EN 10025-2 grades. For example, most VL grades are killed and fine grain treated. This is the case only for the J2+N and K2+N grades in EN 10025-2.

The delivery condition is specified as a function of thickness for all VL grades, while this is either optional or at the manufacturer's discretion in EN 10025-2.

The steel manufacturing process is also at the manufacturer's option in EN 10025-2, while only the electric process or one of the basic oxygen processes is generally allowed according to the DNV GL standard.

Because EN 10025-3 specifies requirements for fine grain treatment, the EN 10025-3 grades are in general better grades than corresponding grades listed in EN 10025-2 and can be considered equivalent with the corresponding VL grades.

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4.2.4.9 Within each defined strength group, different steel grades are given, depending on the required impact toughness properties. The grades are referred to as A, B, D, E, and F for normal weldability grades and AW, BW, DW, and EW for improved weldability grades as defined in [Table 4-3](#).

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.

4.2.4.10 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 up to a specified minimum yield stress of 500 MPa.

4.2.4.11 VL grades with improved weldability are comparable to steel grades according to EN 10225. If steel grades according to EN 10225 will be used instead of VL grades the equivalence of the substitute steel shall be proven.

Table 4-3 Applicable steel grades

Strength group	Test temperature (°C)	VL grade		
		Normal weldability	Improved weldability	Offshore grades
NS	-	A	-	-
	0	B ¹⁾	BW	-
	-20	D	DW	-
	-40	E	EW	-
HS	0	A	AW	-
	-20	D	DW	-
	-40	E	EW	-
	-60	F	-	-
EHS	0	A	-	AO
	-20	D	DW	DO
	-40	E	EW	EO
	-60	F	-	FO

Strength group	Test temperature (°C)	VL grade		
		Normal weldability	Improved weldability	Offshore grades
Note:				
1) Charpy V-notch tests are required for thickness above 25 mm but are subject to agreement between the contracting parties for thicknesses of 25 mm or less.				

4.2.4.12 The grade of steel to be used shall in general be selected according to the service temperature and the thickness for the applicable structural category as specified in Table 4-4. The steel grades in Table 4-3 are VL grade designations.

4.2.4.13 In case the required grade of steel to be used comes out as VL grade F according to Table 4-3 the requirement for the grade of steel can be relaxed to VL grade E when the service temperature is -20°C or higher. In this case, the test temperature can be increased from -60°C to -40°C according to Table 4-3.

4.2.5 Maximum admissible plate thickness and test temperature

Table 4-4 Thickness limitations (mm) of structural steels for different structural categories and service temperatures (°C)

	Strength group	Test temperature (°C)	Maximum thickness (mm) at stated service temperature					
			$\geq 10^{\circ}\text{C}$	0°C	-10°C	-20°C	-25°C	-30°C
Secondary	NS	-	35	30	25	20	15	10
		0	70	60	50	40	30	20
		-20	150	150	100	80	70	60
		-40	150	150	150	150	120	100
	HS	0	60	50	40	30	20	15
		-20	120	100	80	60	50	40
		-40	150	150	150	150	120	100
		-60	150	150	150	150	*)	*)
	EHS	0	70	60	50	40	30	20
		-20	150	150	100	80	70	60
		-40	150	150	150	150	120	100
		-60	150	150	150	150	*)	*)
Primary	NS	-	30	20	10	N.A.	N.A.	N.A.
		0	40	30	25	20	15	10
		-20	70	60	50	40	35	30
		-40	150	150	100	80	70	60
	HS	0	30	25	20	15	12.5	10
		-20	60	50	40	30	25	20

	Strength group	Test temperature (°C)	Maximum thickness (mm) at stated service temperature					
			≥ 10°C	0°C	-10°C	-20°C	-25°C	-30°C
		-40	120	100	80	60	50	40
		-60	150	150	150	150	*)	*)
	EHS	0	35	30	25	20	17.5	15
		-20	70	60	50	40	35	30
		-40	150	150	100	80	70	60
		-60	150	150	150	150	*)	*)
Special	NS	-20	35	30	25	20	17.5	15
		-40	70	60	50	40	35	30
	HS	0	15	10	N.A.	N.A.	N.A.	N.A.
		-20	30	25	20	15	12.5	10
		-40	60	50	40	30	25	20
		-60	120	100	80	60	50	40
	EHS	0	20	15	10	N.A.	N.A.	N.A.
		-20	35	30	25	20	17.5	15
		-40	70	60	50	40	35	30
		-60	150	150	100	80	70	60

*) For service temperatures below -20°C, the limit shall be specifically considered.
N.A. = no application
In order to obtain thickness limitation for intermediate service temperature, linear interpolation may be used.

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

Guidance note:

The grade of steel to be used for thickness less than 10 mm and/or service temperature above 10°C may be specially considered in each case.

For submerged offshore structures, i.e. for structures below LAT-1.5 m, such as in the North Sea, the service temperature will be somewhat above 0°C (typically 2°C) and special considerations may be made in such cases.

If a flange is designed with a flange neck, the minimum distance between weld and flange rounding should be 0.5 times the plate thickness, but at least 5 mm.

Guidance on material selection for flanges with a weld neck may also be found in DIBt:2012, Section 13.4.

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4.2.5.2 Welded steel plates and sections of thickness exceeding the upper limits for the actual steel grade as given in [Table 4-4](#) shall be evaluated in each individual case with respect to the fitness for purpose of the welds. The evaluation should be based on fracture mechanics testing and analysis, e.g. in accordance with BS 7910.

4.2.5.3 For regions subjected to compressive and/or low tensile stresses, consideration may be given to the use of lower steel grades than stated in [Table 4-4](#).

4.2.5.4 The susceptibility of the steel to hydrogen-induced stress cracking (HISC) shall be specially considered when used for critical applications. See also [4.16] and DNVGL-RP-0416.

4.2.5.5 The grade of steel to be used shall in general be selected such that there will be no risk of pitting damage.

4.2.6 Selection of bolt materials

4.2.6.1 The bolt property class for steel structures and interfaces between tower and concrete foundation part shall be designed with the equivalent maximum bolt property class of 10.9, see ISO 898-1 in order to avoid the risk of hydrogen embrittlement.

4.2.6.2 The bolt property class for offshore steel structures below HAT shall be designed with the equivalent maximum bolt property class 8.8, see ISO 898-1.

4.2.6.3 The chemical composition and charpy values for structural bolts shall be in accordance with ISO 898-1.

4.2.6.4 Marking symbols shall indicate the property class of the respective bolt set. Bolt set components may be used from one manufacturer only. Using single bolt set components from different manufacturers is not allowed, see EN 14399.

4.2.6.5 Bolt threads may be rolled before or after heat treatment, followed by a thermal coating process. This thermal coating process may be e.g. hot-dip galvanizing. Electrolytically galvanized corrosion protection is not allowed.

4.2.6.6 Certificates for bolt materials shall be available as required in the relevant execution standard and follow Table 4-5. Structural bolts shall be categorized in a similar way as welded structures.

4.2.7 Selection of welding consumables and filler material

4.2.7.1 Welding consumables and filler material shall be chosen in a proper way. This implies that the material properties specified for the base material shall be achieved for the joint and the related transition to the base material. For details, reference is made to DNVGL-OS-C401 Ch.2 Sec.4 in particular.

4.2.7.2 For high strength and extra high strength steels welding consumables with low-hydrogen content shall be used.

4.2.7.3 The selection of filler material shall be in accordance with DNVGL-OS-B101 Ch.2 Sec.5.

4.2.7.4 If the structure is designed and analysed according to Eurocode the welding consumables have to fulfil the requirements listed in EN 1090-2 Section 5, Table 5.

4.2.8 Material certificates

4.2.8.1 Steel materials and products shall be delivered with inspection documents as defined in EN 10204 or in an equivalent standard. Unless otherwise specified, material certificates according to Table 4-5 shall be presented.

Table 4-5 Material certificates

<i>Structural category</i>	<i>Material certificate (EN 10204)</i>
Special	3.2
Primary	3.1
Secondary	2.2

4.3 Ultimate limit states design – general

4.3.1 General

4.3.1.1 This subsection gives provisions for checking the ultimate limit states for typical structural elements used in wind turbine steel support structures.

4.3.1.2 The ultimate strength capacity of structural elements in yielding and buckling shall be assessed using a rational and justifiable engineering approach.

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

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

4.3.1.5 Onshore structures will typically be protected against corrosion by coating; see also [4.16] and DNVGL-RP-0416.

4.3.2 Corrosion allowance for offshore structures

4.3.2.1 Prediction of structural capacity of offshore structures shall be carried out with due consideration of capacity reductions which are implied by the corrosion allowance; see also [4.16] and DNVGL-RP-0416.

4.3.2.2 The increase in wall thickness for a structural component, added to allow for corrosion, shall be disregarded in the calculation of the structural capacity of the component.

Guidance note:

Structural design of offshore structures in the ULS may be based on a steel wall thickness equal to the nominal thickness reduced by the corrosion allowance over the full service life. The full service life is defined as the sum of (1) the time between installation of the support structure and installation of the wind turbine, (2) the time between installation and operation and (3) the subsequent operation time for the wind turbine.

For primary steel structures in the splash zone, the corrosion allowance can be calculated from the corrosion rates specified, see also [4.16] and DNVGL-RP-0416.

The 2 mm corrosion allowance often applied for replaceable secondary structures in the splash zone is usually not sufficient for a minimum 20-year service life. For boat bumpers whose coating may be vulnerable to damage, such as peel-off and scrape-off caused by approaching supply vessels, a larger corrosion allowance than 2 mm should be considered. For design of boat bumpers current practice is a corrosion allowance between 2 and 4 mm, depending on the quality of the coating system and the risk for vessels that will scratch the coating. In general, the need for corrosion allowance in design of replaceable secondary structures should be balanced against the desirability of replacing such structures.

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4.3.3 Structural analysis

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

4.3.3.2 When plastic or elastic-plastic analyses are used for structures exposed to cyclic loading, e.g. wind turbine loads and 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 (low-cycle fatigue). 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 in the ULS for non-cyclic loads.

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

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

4.3.3.5 When plastic analysis and/or plastic capacity checks are used (cross-section type I and II, according to [App.B](#)), 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.

4.3.3.6 Cross-sections of beams are divided into different types dependent on their ability to develop plastic hinges. A method for determination of cross-sectional types is given in [App.B](#).

4.3.4 Ductility

4.3.4.1 It is a fundamental requirement that all failure modes are sufficiently ductile such that the structural behaviour will be in accordance with the anticipated model used for determination of the responses and such that potential interference between failure modes in ULS and FLS is avoided.

4.3.4.2 For ductile structural steels a detailed analysis of the local impact on the fatigue strength shall be performed if the ULS analysis shows plastic strains in excess of 1% based on linear first order analysis using a representative non-linear stress-strain relation. Plastic strains should in general only occur locally. Regarding plastic strains determined based on plastic or elastic-plastic analyses see [\[4.3.3.2\]](#).

4.3.4.3 Regardless of the analysis method applied, design procedures will in general not capture the true structural behaviour. Still, 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 they shall be verified to have excess resistance compared to ductile modes and in this way protect the structure from brittle failure.

4.3.4.4 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.

4.3.5 Yield check

4.3.5.1 Structural members for which excessive yielding is a possible mode of failure shall be investigated for yielding.

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

4.3.5.3 Regarding limitation of plastification, see [4.3.4.2].

Guidance note:

In cases where linear elastic analysis renders larger zones or peaks of stresses considerably beyond the yield limit, it is necessary to perform an elastic-plastic finite element analysis of a representative detail of subject part of the structure.

Principles for FE-modelling and -analysis of local plastic deformations are given in DNVGL-RP-C208.

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

4.3.5.4 Yield checks may be performed based on net sectional properties.

4.3.5.5 For checks of resistance of welded connections, see [4.10].

4.3.6 Buckling

4.3.6.1 Elements of the member cross-section not fulfilling requirements to cross-section type III (elastic section, see App.B) need to be checked for local buckling.

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

4.3.6.3 Residual stresses and initial imperfections (local or global, geometric or material-wise) shall all be accounted for.

Guidance note:

In a purely analytical approach, an equivalent measure for all such imperfections are typically accounted for through fabrication requirements combined with simple, empirical functions of the relative slenderness, which form a lower bound transition between the (slenderness-independent) simple case of yielding and the classic case of Euler-buckling, where the slenderness is high and imperfections, whatever their nature, are of only a negligible impact on the buckling resistance.

For shell buckling as well as for member buckling, the methods advised in the following sections generally account for imperfections through a simple combination of requirements to fabrication tolerances and slenderness-dependent buckling reduction factors.

For member buckling in particular, the transitions functions (buckling curves) also depend on the profile of the member under consideration.

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

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

4.3.6.5 Buckling failure modes, which combine modes of local buckling with modes of global buckling, shall be appropriately considered.

Guidance note:

A verification of the structural safety against loss of stability in such combined modes is particularly relevant in cases of tubular turbine towers and monopile foundations. In said cases, DNVGL-RP-C202 may be referred for the analysis.

If the structure is designed and analysed according to Eurocode, detailed buckling analysis should be carried out according to EN 1993 series.

In principle, plastification has to be strictly limited to small regions as defined in [4.3.4.2]. For this reason if numerically assisted buckling safety verification is used, the plastic reference resistance [RRpl (plastic value)] has to be substituted by the characteristic reference resistance [RRk (characteristic value)] assuming that the material is still within elastic limit.

The imperfections considered in the design should cover the potential imperfections that are expected to occur during fabrication and installation.

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

4.4 Ultimate limit states - shell structures

4.4.1 General

4.4.1.1 The specifications in present section apply to shell structures, specifically circular cylindrical and conical shells.

4.4.1.2 The strength and stability of shell structures may be checked according to DNVGL-RP-C202 or EN 1993-1-6.

Guidance note:

DNVGL-RP-C202 is applicable for analysis of circular cylindrical shells of uniform section, whether stiffened or not, subject to combined meridional, hoop and in-plane shear stresses. Particularly for stiffened shells, DNVGL-RP-C202 also advises methods for the verification of strength and stability of the stiffeners themselves.

EN 1993-1-6 is applicable for analysis of unstiffened circular cylindrical shells of uniform section for verification of the safety against local buckling likewise under combined meridional, hoop and in-plane shear stresses.

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

4.4.1.3 Wherever openings in a shell structure are designed, their influence on the strength and stability of the structure shall be considered in the design analysis.

Guidance note:

When openings are designed in shells, e.g. in tubular towers and monopile foundations, the Velickov-approach for computation of the meridional shell buckling strength may be considered:

For segments of the shell where circumferentially edge-stiffened openings without additional longitudinal stiffeners are designed ("collar stiffeners", see Figure 4-1) the buckling safety analysis may in simplification be performed along the same procedures as for an unmodified tower shell, provided a reduced value of the critical meridional buckling stress is used instead of the basic shell value.

If DNVGL-RP-C202 is used, the reduced design value of the shell buckling strength shall be taken as

$$f_{ksd, reduced} = C_1 \cdot f_{ksd}$$

where

f_{ksd} is the design shell buckling strength for the unmodified shell according to DNVGL-RP-C202 [3.1]

$C_1 = A_1 - B_1 (r/t)$ is a reduction factor accounting for the influence of the opening, with A_1 and B_1 according to Table 4-6 and with r and t being the radius and thickness, respectively, of the shell section at the level of the cut-out, see Figure 4-1.

If EN 1993-1-6 is used, the reduced design value of the critical meridional buckling stress shall be taken as

$$\sigma_{xs, R, d} = C_1 \cdot \sigma_{x, Rd}$$

where C_1 is defined above and

$\sigma_{x, Rd}$ is the design shell buckling strength for the unmodified shell according to EN 1993-1-6

This simplified method of analysis applies to shells and openings where

- the geometric slenderness value is $(r/t) \leq 160$,
- the opening angle is $\delta \leq 60^\circ$, and
- the aspect ratio of the opening dimensions is $h_1 / b_1 \leq 3$,

where the opening angle δ and the opening dimensions h_1 and b_1 are defined in Figure 4-1 and refer to the cut-out of the tower wall itself, i.e. without consideration of edge-stiffener.

Furthermore, application of the method provides that the opening edge-stiffener

- is arranged centrally about the wall mid-plane at the opening edges (see Figure 4-1 b),
- is of a uniform profile around the entire opening or is considered being so with a section which is equal to its smallest cross-section,
- is of a section with an area equal to at least one-third of the cut-out shell section area, and
- is of a section which individual parts all comply with the limit for local slenderness (c/t -values) which are specified for non-thin-walled sections in EN 1993-1-1 Table 5.2.

Table 4-6 Coefficients A_1 and B_1

	Steel grade S 235		Steel grade S 355	
	A	B	A	B
$\delta \leq 20^\circ$	1.00	0.0019	0.95	0.0021
$\delta = 30^\circ$	0.90	0.0019	0.85	0.0021
$\delta = 60^\circ$	0.75	0.0022	0.70	0.0024

	Steel grade S 235		Steel grade S 355	
	A	B	A	B
Notes: 1) δ = opening angle of the shell along the girth. 2) Intermediate values may be interpolated linearly. Extrapolation is not permitted.				

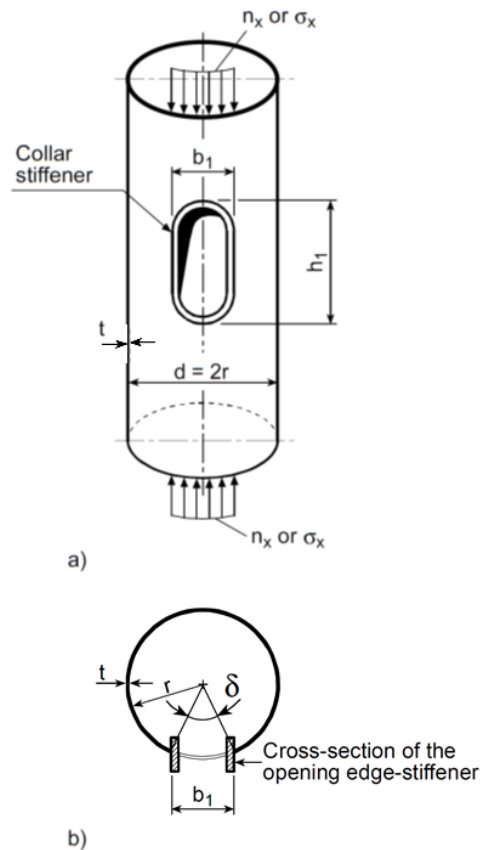


Figure 4-1 Circumferentially edge-stiffened openings

See also [4.5.1.8] regarding verification of safety against loss of stability with due consideration of both shell and beam-column failure criteria.

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

4.4.1.4 Manufacturing tolerances specified in the associated fabrication standard shall be considered in both design and analysis.

4.4.1.5 A buckling verification using FEM is acceptable under consideration of both imperfections and non-linear effects as necessary. The imperfections shall be considered in their relevant locations and in relevant combinations of them. FEM results shall be verified towards empirical or test data.

4.4.1.6 For interaction between shell buckling and beam-column action, see [4.5].

4.4.1.7 If DNVGL-RP-C202 is applied for verification of the buckling resistance of the shell, the material factor shall be:

$$\gamma_M = \begin{cases} 1.10 & \text{for } \bar{\lambda}_s < 0.5 \\ 0.80 + 0.60 \bar{\lambda}_s & \text{for } 0.5 \leq \bar{\lambda}_s \leq 1.0 \\ 1.40 & \text{for } \bar{\lambda}_s > 1.0 \end{cases}$$

In this definition, $\bar{\lambda}_s$ is the relative slenderness pertinent to local buckling of the tubular shell under combined axial, bending, hoop and shear stresses, and shall be determined according to DNVGL-RP-C202 [3.2].

4.4.1.8 The material factor according to Table 4-7 shall be used if EN 1993-1-6 is referred for the verification of the buckling resistance of the shell.

Table 4-7 Material factors used with EN 1993-1-6

Type of calculation	Material factor ¹⁾	Value
LS3 ²⁾ , buckling resistance	γ_{M1}	1.10
¹⁾ Symbols according to EN 1993-1-1		
²⁾ Symbols according to EN 1993-1-6		

4.5 Ultimate limit states - tubular members, tubular joints and conical transitions

4.5.1 Tubular members

4.5.1.1 Tubular members shall be checked according to recognised standards and consider the possible limits on the resistance of the cross section and the stability of the member due to shell buckling. Standards for the strength of tubular members typically have limitations with respect to the D/t ratio and with respect to the effect of hydrostatic pressure.

Guidance note:

The influence of local buckling phenomena on the strength and stability of tubular members is characteristic of thin-walled tubulars of type-IV sections, for which the geometric slenderness parameter, D/t , exceeds $90\epsilon^2$, where $\epsilon = \sqrt{235/f_y}$ and f_y designates the characteristic yield stress of the material (unit N/mm²).

See App.B regarding cross-section types, particularly Table B-3 therein which defines the intervals of the shell slenderness D/t that characterise the four types of tubular member cross sections when subject to axial load and bending.

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

4.5.1.2 The following standards are relevant for checking strength and stability of tubular members: DNVGL-RP-C202, EN 1993-1-1 and EN 1993-1-6, ISO19902 (applicable for D/t < 120) or NORSOK N-004 (applicable for D/t < 120).

Guidance note:

DNVGL-RP-C202 is applicable for analysis of circular cylindrical shells of uniform section, whether stiffened or not, subject to both meridional and hoop stressing in combination with in-plane shear. It may also be applied for analysis of interaction between modes of local shell buckling and flexural buckling of members of uniform section subject to constant axial load combined with bending and external pressure. If DNVGL-RP-C202 is taken as the basis for analysis of members that are of non-uniform section and/or subject to variable axial loads, its limitations need be circumvented by using adequate conservative assumptions.

EN 1993-1-1 is applicable for the stability analysis of tubular members of uniform sections of type I, II and III subject to constant axial loads in combination with bending, but not with hydrostatic pressure. For thin-walled tubulars with cross-section of type IV, EN 1993-1-1 refers to EN 1993-1-6, which applies to analysis of unstiffened circular cylindrical shells, also of uniform sections, for verification of the safety against local buckling under combined axial, hoop and in-plane shear stresses. If EN 1993 is used for the stability analysis of thin-walled tubular members the mentioned limitations have to be considered in a conservative way.

The methods advised in NORSOK N-004 and ISO 19902 are identical. They apply for the stability analysis of unstiffened tubular members of uniform section, subject to constant axial load in combination with bending and external pressure. Particularly, the methods advised in these standards also cover the verification of safety against loss of stability for members of type-IV section where failure modes involve combinations of member flexural buckling and local shell buckling. Thus, NORSOK N-004 and ISO 19902 may be applied for the analysis of tubulars in typical jacket structures, however not for the analysis of tubular towers and monopiles of ordinary designs; see also [4.5.1.7].

All the above-mentioned standards cover actions of shear, EN 1993-1-1, NORSOK N-004 and ISO 19902 however do not consider these being of any interactive influence on member stability, only on the resistance of member sections.

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

4.5.1.3 The basic material factor γ_M for tubular structures is 1.10.

Guidance note:

Depending on the standard used for the check of tubular members, the material factor may vary as a function of the relative shell slenderness of the member considered, see below.

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

4.5.1.4 If DNVGL-RP-C202 is used, the material factors γ_M defined in [4.4.1.7] apply.

4.5.1.5 If EN 1993-1-1 and/or EN 1993-1-6 is used, the material factors γ_M given in Table 4-8 apply.

4.5.1.6 If NORSOK N-004 or ISO 19902 is used, the material factor γ_M defined as:

$$\gamma_M = \begin{cases} 1.10 & \text{for } \bar{\lambda}_s < 0.5 \\ 0.80 + 0.60 \bar{\lambda}_s & \text{for } 0.5 \leq \bar{\lambda}_s \leq 1.0 \\ 1.40 & \text{for } \bar{\lambda}_s > 1.0 \end{cases}$$

shall apply. In this definition, $\bar{\lambda}_s$ is the relative slenderness pertinent to local buckling of the tubular member under combined axial, bending and hoop stresses, to be determined according to NORSOK N-004 Sec. 6.3.7. According to NORSOK N-004 and ISO 19902, actions of shear and torsion shall be checked as being of no interactive influence on member stability, only on the resistance of member sections. For said checks, the value of the material factor shall be taken as $\gamma_M = 1.10$.

4.5.1.7 Tubular towers and monopiles shall be considered tubular structures.

Guidance note:

Towers and monopiles are typically composed of several different tubular or vaguely conical segments, most of which normally are thin-walled (type-IV).

Thus, the section of a typical tubular tower and/or monopile is non-uniform, varying in consequence of both the segmentation of wall thickness and the (typically rather low) gradient in diameter change. See also [4.5.3.4].

Furthermore, the axial load, which is the structural response of the gravity of the rotor-nacelle assembly and the tower/monopile structure itself, including internals and attachments, also varies along the structural axis.

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

4.5.1.8 For tubular towers and monopiles, the safety against loss of stability shall be verified with due consideration of both shell and beam-column failure criteria.

Guidance note:

Analysis methods, which (one way or the other) include both column-related imperfections and settlements in the soil in the computation of the global load response and which indirectly account for the influence of shell imperfections in the determination of the resistance to shell buckling, comply with this requirement if the global load response calculation also include those second order contributions to the bending moment which result from the compressive axial loads acting in combination with the reversible lateral deflections of the structure (P- Δ effects).

Note that for offshore installations, both tower, substructure and foundation need be included in the model if a consistent account for column-related imperfections and settlements in the soil and for second order (P- Δ) effects shall be achieved.

Other generally accepted safe methods for a simplified combination of the criteria for the structure's failure as a shell or a beam-column may alternatively be applied.

Regardless, the documentation of the design analysis needs to clearly explain the methods by which the settlements in soil and the imperfections - both the column-related and the shell-related imperfections - are considered. See also [3.5.3.1] and [3.10.2.2].

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

4.5.2 Tubular joints

4.5.2.1 Tubular joints may be designed and checked according to Norsok N-004 Section 6.4, ISO 19902 Section 14 or other agreed standards.

Guidance note:

Norsok N-004 and ISO 19902 are both applicable for design and strength analysis of simple tubular joints characterised by chords of moderate shell slenderness, limited angles between braces and chords, moderate proportions between brace and chord dimensions and limited overlap between braces, if any.

The design rules and the procedures for joint classification and computation of resistances, which are advised in Norsok N-004 and ISO 19902 for simple tubular joints, are essentially identical. However, parametric equations for strength factors and influence of chord action differ between the two standards.

For the analysis of tubular joints of geometries that fall beyond even a liberal categorisation as 'simple' - this for example being due to gussets, diaphragms, stiffening, grouting or complex overlapping/interaction of principal braces in different planes - application of calibrated finite element methods is advised.

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

4.5.2.2 The material factor γ_M for tubular joints is 1.10.

4.5.3 Conical transitions

4.5.3.1 Concentric conical transitions may be checked by the methods advised in Norsok N-004 Section 6.5, ISO 19902 Section 13.6 or other agreed standards.

Guidance note:

The methods advised in NORSOK N-004 and ISO 19902 are essentially identical. They apply for the analysis of concentric unstiffened conical segments of uniform thickness and slope angle α less than 30° , subject to axial loads in combination with bending and external pressure. Particularly, the methods advised in these standards also cover the verification of safety against loss of local stability for conical segments of thin-walled type-IV sections.

The slope angle α is defined as half the apex angle of the theoretically complete cone.

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

4.5.3.2 The material factor γ_M for conical transitions is 1.10.

4.5.3.3 If NORSOK N-004 or ISO 19902 is used the material factors γ_M given in [4.5.1.4] apply when computed for the geometry of the wider end of the conical segment and for the axial, bending and hoop stresses acting there.

In the determination of γ_M , the characteristic local buckling strength of the conical segment shall be computed as for a circular cylinder of same thickness but equivalent diameter D_e defined by $D_e = D_{actual} / \cos(\alpha)$. See also NORSOK N-004 Sec.6.5.

4.5.3.4 Concentric conical segments of uniform thickness t and a limited slope angle α may be analysed with approximate accuracy by use of the methods for tubular members, which are referred to in [4.5.1], without modifications.

Guidance note:

It is recommended to limit the use of this simpler approach to cases where the slope angle α is small enough for $\cos \alpha$ to be taken equal to unity as an acceptable approximation, e.g. $\alpha < 8^\circ$.

Regardless of the conical segment being analysed in the simple approach or not, the local stress condition at the junctions should always be analysed separately. See also NORSOK N-004 Sec.6.5.

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

4.6 Ultimate limit states – non-tubular beams, columns and frames

4.6.1 General

4.6.1.1 The design of non-tubular beams, columns and frames shall be checked according to recognised standards and shall consider the possible limits on the resistance of the cross-section and the stability of the member due to local buckling.

Guidance note:

Cross-sections of members are categorised into four different types (compact (I), semi-compact (II), elastic (III) and thin-walled (IV)) depending on their ability to develop plastic hinges (characteristic of sections entirely of type I) and/or to resist local buckling in one or all its parts.

The influence of local buckling phenomena on the strength and stability of beams, columns and frames is characteristic of thin-walled members of type-IV sections.

See App.B regarding categorisation of cross-sections in types I, II and III. Thin-walled sections are such for which one or all its parts - webs, flanges, stiffeners - fall beyond the limits of categorisation as type III.

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

4.6.1.2 The following standards are relevant for checking strength and stability of non-tubular members, particularly members of thin-walled type-IV sections: DNV-RP-C201, EN 1993-1-1 and EN 1993-1-5. DNV-RP-C201 is applicable for the analysis of buckling of plate panels, whether stiffened or not, by conventional methods.

EN 1993-1-1 is applicable for the stability analysis of members of ordinary uniform sections subject to constant axial loads in combination with biaxial bending, regardless of the section type. Actions of shear and torsion are considered of no interactive influence on member stability, only on the resistance of cross-sections. Particularly for members of type IV sections, EN 1993-1-1 may also be used for the stability analysis of non-circular cylindrical members in failure modes that combine member flexural buckling and/or lateral torsional buckling with local plate buckling. For thin-walled tubular members, EN 1993-1-1 refers to EN 1993-1-6; see also [4.5].

EN 1993-1-5 applies particularly for the analysis of plated structural elements, typically forming parts of members with slender welded sections and potentially including stiffeners. Influences of both shear lag and local buckling are accounted for in the methods advised for verification of plate and member design.

All the above-mentioned standards apply the principle of effective widths and effective cross-sections in their methods advised for stability analysis of thin-walled plates and members.

4.6.1.3 The material factor γ_M for non-tubular beams, columns and frames is 1.10.

4.6.1.4 If the methods in DNV-RP-C201 are applied, the material factor γ_M shall be taken as 1.10.

4.6.1.5 If EN 1993-1-1 and/or EN 1993-1-5 is used the material factors according to Table 4-8 apply.

Table 4-8 Material factors used with EN 1993-1-1 and EN 1993-1-5

Type of calculation	Material factor ¹⁾	Value
Resistance of type I, II or III sections	γ_{M0}	1.10
Resistance of type IV sections	γ_{M1}	1.10
Buckling resistance	γ_{M1}	1.10
¹⁾ Symbols according to EN 1993-1-1 and EN 1993-1-5		

4.7 Ultimate limit states – plate structures

4.7.1 General

4.7.1.1 The requirements in this subsection pertain to welded structural parts, which are composed of plates subject to in-plane or transverse loads, or combinations thereof.

Guidance note:

Plated structural components, which can be classified as members, should be designed and analysed according to [4.6].

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

4.7.1.2 The material factors according to Table 4-9 shall be used if EN 1993-1-5 and/or EN 1993-1-7 are used for calculation of structural resistance.

Table 4-9 Material factors used with EN 1993-1-5 and EN 1993-1-7

Type of calculation	Material factor ¹⁾	Value
Plate buckling and interaction between plate and column buckling	γ_{M0}	1.10
Shear resistance	γ_{M1}	1.10

Type of calculation	Material factor ¹⁾	Value
¹⁾ Symbols according to EN 1993-1-5		

4.7.1.3 The material factors according to [Table 4-10](#) shall be used if EN 1993-2 is used for calculation of structural resistance.

Table 4-10 Material factors used with EN 1993-2

Type of calculation	Material factor ¹⁾	Value
Resistance of cross-sections to excessive yielding including local buckling	γ_{M0}	1.10
Resistance on members to instability assessed by member checks	γ_{M1}	1.10
Resistance of cross-sections in tension to fracture	γ_{M2}	1.25
¹⁾ Symbols according to EN 1993-2		

4.8 Ultimate limit states – lattice structures, trusses and joints

4.8.1 Scope

4.8.1.1 The requirements in this subsection apply to structures, which are designed and analysed as triangulated structures.

Guidance note:

Non-triangulated structures should be considered as frames and be designed and analysed according to sections [\[4.5\]](#) through [\[4.7\]](#) above.

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

4.8.1.2 The structural response should be determined by way of elastic global analysis. Otherwise the requirements given in [\[4.3.3.4\]](#) shall be considered.

4.8.1.3 The global analysis of the structural response may assume that certain or all members, both flanges and trusses, are effectively pinned, i.e. not including moments at the connections.

Guidance note:

The above may not be applicable in case of very stiff pre-loaded connections.

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

4.8.2 Flanges and trusses

4.8.2.1 Capacity checks of members may be performed according to recognised standards such as EN 1993-3-1.

4.8.2.2 The verification of the members' safety shall duly correspond to the assumptions made for the determination of the structural response.

Guidance note:

The verification should be based on values of buckling lengths and bending moments fully corresponding to the assumptions for the global analysis and the member loads thus determined.

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

4.8.2.3 The material factors according to [Table 4-11](#) shall be used if the methods of analysis given in EN 1993-3-1 are referred for the verification of the structural resistance.

Table 4-11 Material factors used with EN 1993-3-1

<i>Type of calculation</i>	<i>Material factor ¹⁾</i>	<i>Value</i>
Yielding resistance of members	γ_{M0}	1.10
Buckling resistance of members	γ_{M1}	1.10
Resistance of net sections at bolt holes	γ_{M2}	1.25
¹⁾ Symbols according to EN 1993-1-1		

4.8.3 Joints

4.8.3.1 Connections shall be designed and analysed according to the provisions made in [\[4.5.2\]](#) for tubular joints and in [\[4.9\]](#) and [\[4.10\]](#) for bolted and welded connections, respectively.

4.8.3.2 The verification of the safety of connections shall appropriately correspond to the assumptions made for the determination of the structural response.

4.9 Ultimate limit states – bolted connections

4.9.1 General

4.9.1.1 The requirements in this subsection pertain to pre-loaded bolted connections, subject to axial loads, shear loads or combinations thereof in the ULS. Bolted connections without pre-loading should not be used for wind turbine support structures.

The specifications particularly cover flange connections in towers and monopiles, anchor bolts in concrete foundations and bolted joints in frame or lattice structures.

If bolted connections are designed as ordinary (bearing type) connections, it shall be demonstrated that potential plastic deformations are within allowable limits. Moreover this type of connections should always be locked e.g. by way of counter-nut or pin.

Guidance note:

Bolted connections in secondary structural members may be designed as pinned connections, i.e. without any pre-loading of the bolts. This may be done following e.g. EN 1993-1-8.

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

4.9.1.2 The requirements assume that the bolt materials are selected according to [\[4.2.6\]](#).

4.9.1.3 The bolted connections shall be designed and preloaded to be slip-resistant at ultimate external loadings.

4.9.1.4 The safety of the bolted connections may be verified according to EN 1993-1-8, in which case the material factors stated in [Table 4-12](#) below shall apply:

Table 4-12 Material factors used with EN 1993-1-8 for bolted connections

<i>Type of resistance assessed</i>	<i>Material factor ¹⁾</i>	<i>Value</i>
Net member section resistance	γ_{M0}	1.10
Tension resistance	γ_{M2}	1.25
Bearing resistance	γ_{M2}	1.25
Slip resistance	γ_{M3}	See [4.9.5]
1) Symbols according to EN 1993-1-8		

4.9.2 Bolt hole geometry

4.9.2.1 Unless specifically designed, normal clearance for fitted bolts shall be assumed.

4.9.2.2 If oversized holes are considered, the impact shall be considered in the design according to EN 1993-1-8.

4.9.2.3 The clearances and nominal sizes of normal bolt holes are defined in [Table 4-13](#).

Table 4-13 Clearance in bolt holes

<i>Clearance type</i>	<i>Bolt diameter d (maximum) mm</i>	<i>Clearance mm</i>	<i>Bolt hole diameter mm</i>
Standard	12 and 14	1	d+1
	16 - 24	2	d+2
	27 - 36	3	d+3
	42 - 48	4	d+4
	56	5	d+5
	64	6	d+6
	72	6	d+6
Oversized	12	3	d+3
	14 - 22	4	d+4
	24	6	d+6
	27	8	d+8

Guidance note:

The bearing resistance of the hole is assumed to be less than 30% for standard clearance for M42 – M72. Otherwise the standard clearance is limited to only 3 mm also for M42 – M72.

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

4.9.2.4 The nominal sizes of short slotted holes for slip resistant connections shall not be greater than given in Table 4-14.

4.9.2.5 If long slotted holes are considered, the impact shall be considered in the design according to EN 1993-1-8.

Table 4-14 Short slotted holes

<i>Maximum size mm</i>	<i>Bolt diameter d (maximum) mm</i>
$(d+1) \times (d+4)$	12 and 14
$(d+2) \times (d+6)$	16 – 22
$(d+2) \times (d+8)$	24
$(d+3) \times (d+10)$	27 and beyond

4.9.2.6 The nominal sizes of long slotted holes for slip resistant connections shall not be greater than given in Table 4-15.

Table 4-15 Long slotted holes

<i>Maximum size mm</i>	<i>Bolt diameter d (maximum) mm</i>
$(d+1) \times 2.5d$	12 and 14
$(d+2) \times 2.5d$	16 - 24
$(d+2) \times 2.5d$	27 and beyond

4.9.3 Preloading of bolts

4.9.3.1 The bolt shall be preloaded in accordance with international recognised standard procedures. Methods for measurement and procedures for maintenance of the bolt tension shall be established as part of the design.

4.9.3.2 For high strength bolts in flanged connections and joints in framed or latticed structures with torque-controlled tensioning, the controlled preloading force in the bolt shall be:

$$F_p \leq 0.7 f_{ub} A_s$$

where:

f_{ub} = characteristic ultimate tensile strength of bolt material

A_s = tensile stress area of bolt (net area in the threaded part of the bolt)

4.9.3.3 For anchor rods (typically custom-designed), the controlled pre-loading force in the rod shall be:

$$F_p \leq 0.7 f_{u,rod} A_{ref}$$

where:

$f_{u,rod}$ = characteristic ultimate tensile strength of anchor rod material

A_{ref} = minimum of tensile stress area of threaded parts (A_s) and nominal section area (A_{nom}) of unthreaded part of rod.

4.9.3.4 If other methods of tensioning are considered this should follow the provisions given in EN 1090-2.

4.9.4 Axially loaded connections

4.9.4.1 The requirements in [4.9.4] pertain to bolted connections subject to axial loads, only.

4.9.4.2 Changes of loads in bolts and surrounding base materials due to the external loading shall be determined based on an elastic model reflecting the (axial) stiffness of the two components.

4.9.4.3 Prying of bolt forces due to deformation of flange plates shall be accounted for.

Guidance note:

For bolted flange connections in e.g. tubular towers and monopiles, the methods of analysis of bolts and flanges, which are described in Petersen, C, *Stahlbau: Grundlagen der Berechnung und baulichen Ausbildung von Stahlbauten /1/* or Seidel, M. *Zur Bemessung geschraubter Ringflanschverbindungen von Windenergieanlagen /2/* may be applied for the computation of loads in the two components in combination with a limitation of flatness deviation. These methods implicitly assume local yielding in the ULS. Computation of loads in the bolt and flange components may also be based on FE-models, adequately representing real geometry of the connection including admissible imperfections and allowable tolerances.

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

4.9.4.4 The response of the connection to the external load shall be fully reversible as otherwise the preloading can be lost. This shall be secured by the SLS check specified in [4.15].

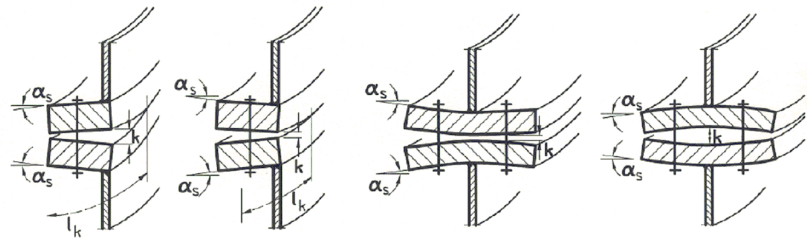
Also, the connection shall be designed so that the resulting compressive stressing of the compressed parts will not be permanently reduced to zero (resulting in a permanent separation of the connected parts) in consequence of the ultimate external tensile load.

Furthermore, the connection shall be designed so that the resulting force in the bolt is not reduced to zero under the ultimate external compressive load as otherwise the preloading can be lost, even if the external load is of a momentary nature.

Guidance note:

Adequate compressive preloading of the flange contact surfaces may be assumed if the flatness deviation per flange (similar to $k/2$ in Figure 4-2) is within the tolerance value of 2 mm over the whole circumference and maximum 1 mm over an angle of up to 30° after completion of the manufacturing process of the tubular segment. It is assumed that flatness deviations k of assembled flanges will not sum up to 4 mm in total as the maximum of each part is not at the same location on the perimeter.

If the tolerance values for the flange gaps are not complied with, suitable measures should be taken. Suitable measures may e.g. include reworking, shimming or filling out the damage-relevant gap spaces before preloading takes place.



a) L-flange,

b) T-flange

Figure 4-2 Ring flange connections in steel towers

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

4.9.4.5 The design tension resistance of the individual bolt, $F_{t,Rd}$, shall be taken as:

$$F_{t,Rd} = \frac{0.9 f_{ub} A_s}{\gamma_{M2}} \text{ for high-strength bolts, and}$$

$$F_{t,Rd} = \frac{0.9 f_{u,rod} A_{ref}}{\gamma_{M2}} \text{ for anchor rods.}$$

Guidance note:

Bolted connections involving bolts with countersunk heads are not recommended.

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

4.9.4.6 The safety criterion for an individual bolt connection subject to pure axial load is:

$$\frac{F_t}{F_{t,Rd}} \leq 1.00$$

F_t designates the resulting design tensile bolt load, this being a function of the preload and the external axial load transferred by the individual connection.

4.9.5 Shear loaded connections

4.9.5.1 The requirements in [4.9.5] pertain to bolted connections subject to shear, only.

4.9.5.2 End, edge and mutual distances between bolts in a shear loaded connection shall in general be designed with minimum values as defined in Table 3.3 in EN 1993-1-8. See also [4.12.4.2].

4.9.5.3 The bolt group, which constitutes the joint, shall be designed so that its design resistances against slip and bearing both are higher than the design ultimate shear load:

$$\sum F_{s,Rd} \geq \sum F_s \quad \text{and} \quad \sum F_{b,Rd} \geq \sum F_s$$

where:

F_s = the design shear load to be transferred by the individual bolt connection

$F_{s,Rd}$ = the design slip resistance of the individual bolt connection

$F_{b,Rd}$ = the design bearing resistance of the individual bolt connection

4.9.5.4 Furthermore, for a shear-loaded connection, which transfers a tensile load in the connected member, the design plastic resistance of the net section of the member shall be higher than its design tensile load:

$$N_{net,Rd} \geq \sum F_s$$

4.9.5.5 Classification of surface friction and calculation of slip resistance for a preloaded bolt shall be performed according to DNVGL-OS-C101 Ch.2 Sec.11.

4.9.5.6 The design bearing resistance of the individual bolt connection, $F_{b,Rd}$, may be determined according to the rules given in Table 3.4 of EN 1993-1-8.

4.9.5.7 The design plastic resistance of the net section of a member transferring a tensile load shall be taken as:

$$N_{net,Rd} = \frac{f_y}{\gamma_{M0}} A_{net}$$

4.9.5.8 Oversized holes in outer plies of slip resistant connections shall be covered by hardened washers.

4.9.5.9 Long slotted holes in outer plies shall be covered by cover plates of appropriate dimensions and thickness. The holes in the cover plate may not be larger than standard holes.

4.9.5.10 The safety criterion for an individual bolt connection subject to pure shear is:

$$\frac{F_s}{F_{Rd}} \leq 1.00$$

where:

$$F_{Rd} = \text{Min} (F_{s,Rd}, F_{b,Rd})$$

4.9.6 Connections subject to combined axial and shear loads

4.9.6.1 The safety criterion for an individual bolt connection subject to combined tension and shear shall be taken as:

$$\frac{F_t}{F_{t,Rd}} \leq 1.00$$

together with:

$$\frac{F_s}{F_{Rd,red}} \leq 1.00$$

where:

$$F_{Rd,red} = \text{Min} \left(\frac{k_s n \mu F_c}{\gamma_{M3}}, F_{b,Rd} \right)$$

F_c designates the resulting design compressive load between the plates connected, this being a function of the preload and the external axial load transferred by the connection. See also paragraph [4.9.5.3]. Any one individual bolt connection in the joint design shall comply with this safety criterion.

4.9.6.2 In addition to the criterion applying to the individual bolt connection, the criterion stated in paragraph [4.9.5.3] shall also apply.

4.10 Ultimate limit states – welded connections

4.10.1 Scope

The requirements in this subsection apply to welded connections.

4.10.2 General requirements for welded connections

4.10.2.1 All types of butt joints should be welded from both sides. It shall be specified that before welding is carried out from the second side, weld errors such as lack of infusion, inclusions, etc., shall be removed at the root by a suitable method.

4.10.2.2 The connection of a plate abutting on another plate in a T- or cross joint may be made as indicated in Figure 4-3.

4.10.2.3 The thickness of the throat t_w (the a-measure) of a fillet weld shall be taken as the normal to the weld surface, as indicated in Figure 4-3 d).

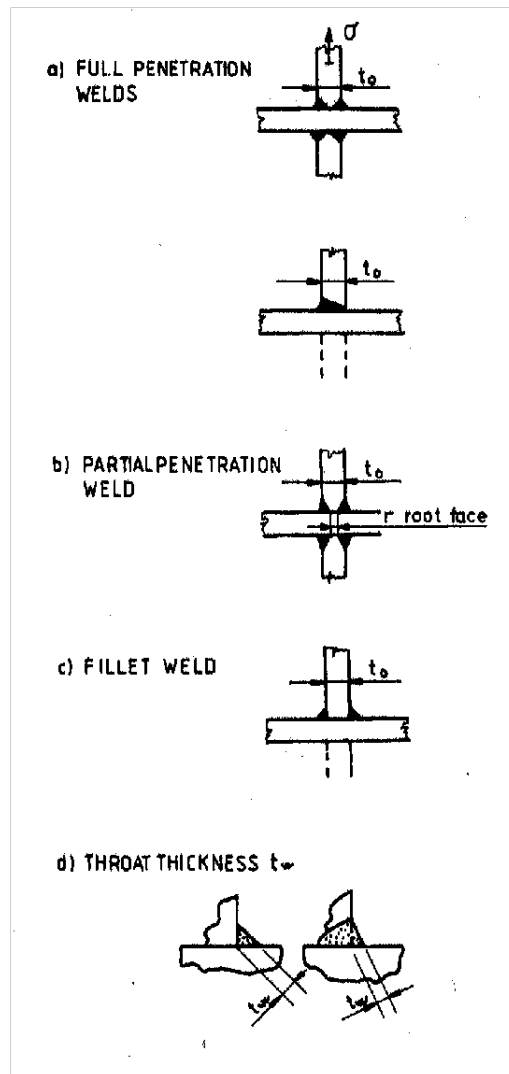


Figure 4-3 Tee and cross joints

4.10.2.4 The type of connection should be adopted as follows:

- Full penetration weld is used for important connections in structures exposed to high stress, especially dynamic, e.g. for special areas and fatigue utilised primary structure, such as tubular joints, plate butt joints, and longitudinal and circumferential welds in tubulars.
- Partial penetration weld is used in connections where the static stress level is high. Acceptable also for dynamically stressed connections, provided the equivalent stress is acceptable.
- Fillet weld is used in 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.

4.10.2.5 The material factor γ_{Mw} for welded connections is given in [Table 4-16](#).

Table 4-16 Material factors γ_{Mw} for welded connections

Limit state	Material factor
ULS	1.25

4.10.2.6 The quality of the welding consumables designed for the welded connections shall be specified in the project in terms of minimum requirements to both yield and ultimate capacity.

4.10.2.7 It is allowed to design for filler materials of different grades (both higher and lower) than the base material, still the verification of the weld size shall correctly refer to the characteristics of the filler material strength.

4.10.2.8 Welds shall be designed with dimensions such that their design strength is not lower than the design strength of the parts joined by them.

4.10.2.9 The distribution of forces in a statically loaded welded connection may be calculated directly based on an assumption of either elastic or plastic behaviour.

4.10.2.10 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, σ_{II} .

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

4.10.2.12 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.

4.10.2.13 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.

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

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

and:

$$\sigma_{\perp d} \leq \frac{f_{uw}}{\gamma_{Mw}}$$

where:

$\sigma_{\perp d}$ = design value of normal stress transverse to throat of weld

$\tau_{\perp d}$ = design value of shear stress on throat of weld, transverse to the direction of the weld axis

$\tau_{II d}$ = design value of shear stress on throat of weld, parallel to the direction of the weld axis

f_{uw} = nominal lowest ultimate tensile strength of the weld filler material

β_w = correlation factor, see Table 4-17

γ_{Mw} = material factor for welds

see also Figure 4-4.

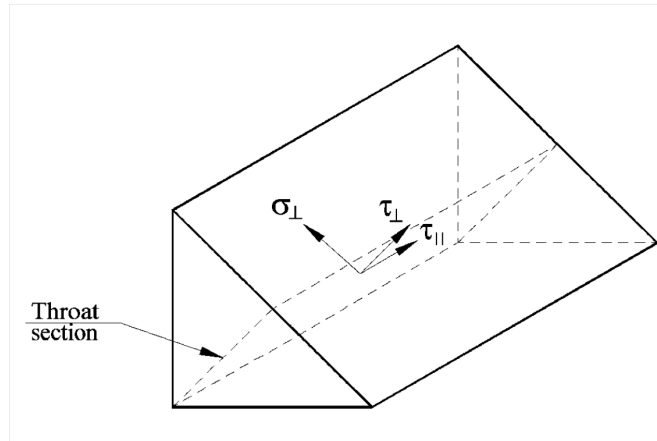


Figure 4-4 Components of fillet weld stress state

Table 4-17 Correlation factor β_w

Lowest ultimate tensile strength f_{uw} (MPa)	Correlation factor β_w
400	0.83
440	0.86
490	0.89
510	0.90
530	1.00
570	1.00

4.10.3 Tubular joints

Welds in tubular joints shall be designed as full penetration welds of filler materials of same or higher strength as the parent steel joined by them.

4.11 Fatigue limit states design – general

4.11.1 Scope

4.11.1.1 In this subsection, requirements are given for design against fatigue failure.

4.11.1.2 Standards and guidelines covering design against fatigue failure may be applied, such as DNVGL-RP-C203 and EN 1993-1-9, together with their associated execution standards. The applied design standard or guideline shall be suitable for the type of structure it shall be applied for, and shall address the corrosive environments to which the structure is exposed. The applied design standard shall be internationally recognized. The applied design standard shall be part of a consistent set of standards covering not only fatigue design, but also fabrication including requirements to non-destructive testing. A general reference is made to DNVGL-RP-C203 for practical details in design and analysis of offshore steel structures.

4.11.1.3 The aim of fatigue design is to ensure that the structure has sufficient resistance against fatigue failure, i.e. that it has an adequate fatigue life. Assessment of fatigue lives is used in fatigue design to fulfil this aim. Assessment of fatigue lives can also form the basis for definition of efficient inspection programs, both during manufacturing and during the operational life of the structure.

4.11.1.4 The resistance against fatigue of a particular structural detail is normally given in terms of an S-N curve. The S-N curve gives the number of cycles to failure N versus the stress range $\Delta\sigma$. The S-N curve is usually based on fatigue tests in the laboratory, see also DNVGL-RP-C203 [2.4].

4.11.1.5 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.

4.11.1.6 The design fatigue life shall also account for the fatigue, which will accumulate during stages of transportation, installation and pre-operation.

4.11.1.7 To ensure that the structure will fulfil the intended function, a fatigue assessment shall be carried out for each individual member and connection subjected to fatigue loading.

Guidance note:

Any element or member of the structure, every welded joint or attachment or other form of stress concentration is potentially a source of fatigue cracking and should be considered individually.

It is utmost important that the stress cycles are determined based on models for the structural response, which realistically reflect the stiffness of both members and joints.

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

4.11.1.8 For offshore structures corrosion allowance shall be taken into account by decreasing the nominal wall thickness in fatigue limit state analyses.

Guidance note:

Fatigue calculations may be based on a steel wall thickness equal to the nominal thickness reduced by half the corrosion allowance over the full service life without de-commissioning phase. The full service life is defined as the sum of (1) the time between installation of the support structure and installation of the wind turbine, (2) the time between installation and operation and (3) the subsequent operation time for the wind turbine.

Corrosion allowance covering a period after the turbine is taken out of service does not need to be accounted for in the fatigue calculation covering (1) through (3) above.

For primary steel structures in the splash zone, the corrosion allowance may be calculated from the corrosion rates specified in DNVGL-RP-0416.

The 2 mm corrosion allowance often applied for replaceable secondary structures in the splash zone is usually not sufficient for a 20-year service life. For boat bumpers whose coating may be vulnerable to damage, such as peel-off and scrape-off caused by approaching supply vessels, a larger corrosion allowance than 2 mm should be considered. In general, the need for corrosion allowance in design of replaceable secondary structures should be balanced against the desirability of replacing such structures.

When corrosion allowance is specified to account for expected metal loss during the operational lifetime, the associated S-N curve free corrosion should be applied. If corrosion protection by e.g. coating is applied, the application of the S-N curve free corrosion may be limited to the part of the lifetime that exceeds the expected lifetime of the corrosion protection.

It is strongly recommended that the structure is protected by the CP system as soon as possible after installation of the structure, if no other corrosion protection such as coating is applied.

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

4.11.2 Characteristic S-N curves

4.11.2.1 The fatigue strength of welded joints is to some extent dependent on plate thickness, this being due to the local geometry of the weld toe in relation to the thickness of the adjoining plates. The thickness effect is accounted for by a thickness-dependent modification of the stress range used to calculate the number of cycles to failure.

This implies that the characteristic S-N relationship for any particular detail under consideration can be taken as:

$$N = \frac{a}{\left(\Delta\sigma \cdot \left(\frac{t}{t_{ref}}\right)^k\right)^m}$$

or:

$$\log_{10} N = \log_{10} a - m \log_{10} \left(\Delta\sigma \cdot \left(\frac{t}{t_{ref}}\right)^k \right)$$

as a modification of the 'basic' S-N curve: $N = \frac{a}{\Delta\sigma^m}$.

where:

N = number of cycles, all of width equal to S, implicating fatigue failure initiated at subject detail,

a = the key identification of the S-N curve, $N = \frac{a}{\Delta\sigma^m}$, relevant for subject detail (a is identified as the intercept of the S-N curve with the N-axis (S=1) in a log-log depiction),

$\Delta\sigma$ = stress range at subject detail in units of MPa

m = the Wöhler exponent (m is identified as the negative inverse slope of the S-N curve in a log-log depiction)

t_{ref} = reference thickness, being

$t_{ref} = 16$ mm for tubular joints,

$t_{ref} = 25$ mm for welded connections other than tubular joints, and

$t_{ref} = 25$ mm for bolts

t = actual thickness through which the potential fatigue crack will grow

t = t_{ref} shall be entered in all cases where $t < t_{ref}$

k = thickness exponent.

The general definition of S-N curves together with S-N curves applicable to a range of typical details in steel structures is given in DNVGL-RP-C203.

The use of the S-N curves in DNVGL-RP-C203 is an option. They can be used when project-specific or manufacturer-specific data are not available for all ranges of applicability otherwise covered by S-N curves.

Guidance note:

In general, the classification of structural details and their corresponding S-N curves in air, in seawater with adequate cathodic protection and in free corrosion conditions, may be taken from DNVGL-RP-C203.

S-N Curves specified for material in air are valid for details, which are located above the splash zone. The in air curves may also be utilised for the internal parts of air-filled members below water and for pile driving fatigue analysis.

The basis for the use of the S-N curves specified in DNVGL-RP-C203 is that a high fabrication quality of the details is present, i.e. welding and non-destructive testing should be in accordance with inspection category I and structural category special according to DNVGL-OS-C401 Ch.2 Sec.7 Table 1.

The free corrosion S-N curves may be used below the waterline for internal surfaces of monopiles.

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

4.11.2.2 The thickness-dependent modification of the stress range in [4.11.2.1] is intended to reflect that the actual size and geometry of the structural component considered are different from what the S-N data (loga and m) are based on: The modification accounts for the effect of different sizes of plates that a potential fatigue crack will grow through. However, there may also be an effect of the weld attachment length in cruciform joints, the weld width in butt welds, and the combined weld thickness and thickness transition length in tubular girth welds. The effect of this measure of length and width, L , is that the actual thickness t in the S-N relationship can be replaced by an effective thickness

$$t_{eff} = \min(14 + 0.66 \cdot L; t) [mm],$$

where L is defined in DNVGL-RP-C203 [2.4.3] for cruciform joints and butt welds.

4.11.2.3 Requirements for use of S-N curves are given in [4.11.5].

4.11.2.4 S-N curves from other standards than DNVGL-RP-C203, such as EN 1993-1-9, may be applied if they fulfil other requirements set forth in this standard. When other standards are applied, it shall be evaluated by the designer that they adequately well capture damage contributions from all relevant load cycles applied to the structure.

Guidance note:

When EN 1993-1-9 is applied as the design standard, the following modifications to S-N curves should be considered:

For components predominantly loaded by normal stresses, the following applies to the S-N curve as per EN1993-1-9:

Region I: slope parameter of the S-N curve $m_1 = 3$, stress cycle numbers $N_i < 5 \cdot 10^6$

Region II: slope parameter of the S-N curve $m_2 = 5$, stress cycle numbers $N_i \geq 5 \cdot 10^6$

For predominantly shear-stress loaded components, the S-N curves of EN 1993-1-9:2005 should be used with a constant slope parameter $m = 5$:

Region I + II: slope parameter of the S-N curve $m = 5$, all numbers of stress cycles

A threshold value of the fatigue strength (cut-off) should not be applied.

The reason for this recommendation lies in the limited experience with high-cycle fatigue loading both in research as well as field experience from existing wind turbine structures. It reflects an approach that has been used frequently in the wind industry in different rules and standards which is considered a conservative design assumption.

A thickness correction factor should be applied in line with the requirement in EN 1993-1-9.

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

4.11.2.5 As a supplement to a fatigue calculation based on an appropriate S-N curve, a calculation of the fatigue life may be based on a fracture mechanics analysis, see DNVGL-RP-C203. A method for fracture mechanics calculations can be found in BS 7910.

4.11.3 Characteristic stress range distribution

4.11.3.1 A characteristic long-term stress range distribution shall be established for the structure or structural component.

4.11.3.2 All significant stress ranges, which contribute to fatigue damage in the structure, shall be considered.

Guidance note:

For multi-axially stressed welded connections between components it is important to consider the complex stress conditions in a realistic manner that allows for a physically meaningful damage accumulation calculation. This is true for both proportional and non-proportional loading, the latter being especially relevant for offshore foundations where the combined loading from wind and wave actions typically are out of phase and often misaligned.

When analysing multi-axial stresses, it is recommended that the dominating (damage-relevant) stress distribution or stress combination be established for the critical regions via consideration of the principal stresses and principal-stress directions. Occasionally, the presence of a dominating load component, or the combination of load components, may lead to a stress condition that is close to uniaxial. In such cases, this may allow a possible simplification that is appropriate for the problem.

For proportional loading, fatigue should be based on the principal stresses. In cases where the direction vector of the principal stress is approximately transverse to the weld seam and does not change significantly over time, the normal-to-weld stress may be used. If the direction vector varies significantly, the other principal stresses need to be analysed as well.

For non-proportional loading characterized by a significant continuous variation of the principal stress direction during a load case, it will likely be difficult to pinpoint just one (or even a few) dominant principal-stress direction, yet this needs to be done based on an analysis and physical interpretation of the individual stress sequence.

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

4.11.3.3 Stress ranges caused by wind loading shall be established from site-specific wind statistics. Stress ranges caused by wind loading shall be established under due consideration of the actual alignment of the rotor axis of the wind turbine relative to the direction of the wind. Stress ranges arising during fault conditions where a yaw error is present need to be considered.

4.11.3.4 Stress ranges caused by wave loading shall be established from site specific wave statistics. Discrete wave statistics can be applied for this purpose and usually imply that the number of waves are specified from eight different compass directions in one-meter wave height intervals. For wave heights between 0 and 1 m, a finer discretisation with 0.2 m wave height intervals is recommended in order to enhance the accuracy of the fatigue damage predictions for the loading arising from waves heights in this range.

4.11.3.5 The choice of wave theory to be applied for calculation of wave kinematics shall be made according to DNVGL-ST-0437. The wave theory depends much on the water depth. For water depths less than approximately 15 m, higher order stream function theory renders the most reliable results and is therefore to be applied. For water depths in excess of approximately 30 m, Stokes 5th order theory shall be applied.

4.11.3.6 Stress ranges caused by the operation and control of the wind turbine shall be included. They include stress ranges owing to drive train mechanical braking and transient loads caused by rotor stopping and starting, generator connection and disconnection, and yawing loads.

4.11.3.7 Stress ranges caused by transportation shall be considered.

4.11.3.8 For driven steel piles, stress ranges caused by the driving of the piles shall be included. Likewise, for piles installed by vibration, stress ranges caused by the vibration shall be included.

4.11.3.9 In particular, stress ranges caused by environmental conditions in the pre-commissioning phase (e.g. from vortex induced vibrations) shall be included.

4.11.3.10 Whenever appropriate, all stress ranges of the long-term distribution shall be multiplied by a stress concentration factor (SCF). The SCF depends on the structural geometry. SCFs can in some cases be calculated from parametric equations and in general by finite element analysis.

Guidance note:

In wind farms, where designs of joints or structural details are typically repeated in high numbers in identical or near-identical support structures, requirements to cost-effectiveness makes an accurate assessment of the SCF particularly relevant. In such cases assessment by finite element analysis is recommended.

When parametric equations are used to calculate SCFs for tubular joints, the Efthymiou equations should be applied for T-, Y-, DT- and X-joints, as well as for K- and KT-joints. For details, see DNVGL-RP-C203 App.B.

For multi-planar tubular joints where multi-planar effects cannot be neglected, the SCFs may be determined by a detailed FEM analysis of each joint detail. Alternatively, the SCF may be taken as the largest value found by the parametric computations applied to the joint detail assuming it being a planar Y-, X- or K-joint.

When conical stubs are used, the SCF may be determined by referring to the cone cross-section at the point where the centre line of the cone intersects the outer surface of the chord. For gapped joints with conical stubs, the true gaps should be applied.

A minimum SCF equal to 1.5 should be adopted for tubular joints if no other documentation is available.

In tubular girth welds, geometrical stress increases are induced by local bending moments in the tube wall, created by centre line misalignment from tapering, by fabrication tolerances and by differences in hoop stiffness for tubes of different thickness. Details for calculation of SCFs for tubular girth welds are given in DNVGL-RP-C203. It is recommended that as strict fabrication tolerances as possible is specified for tubular girth welds as a means for minimising the stress concentration factor.

In case another design standard than DNVGL-RP-C203 is applied, such as EN 1993-1-9, the detail categories defined in that standard shall be applied together with the relevant tolerance requirements specified in that same standard.

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

4.11.3.11 For fatigue analysis of regions in base material not significantly affected by residual stresses due to welding or due to cold-forming, the stress ranges may be reduced prior to the fatigue analysis depending on whether the mean stress is a tensile stress or a compressive stress.

Guidance note:

The reduction is meant to account for effects of partial or full fatigue crack closure when the material is in compression. See also DNVGL-RP-C203 [2.5.1].

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

4.11.3.12 For fatigue analysis of regions in welded structural details, which have been subject to post weld heat treatment or for which correspondingly low residual stresses can be documented, the stress ranges may be reduced prior to the fatigue analysis depending on whether the mean stress is a tensile stress or a compressive stress.

Guidance note:

The reduction is meant to account for effects of partial or full fatigue crack closure when the material is in compression. See also DNVGL-RP-C203 [2.5.2].

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

4.11.3.13 Dynamic effects, including dynamic amplification, shall be duly accounted for when establishing the long-term stress range distribution.

4.11.3.14 The stress ranges in the stress range distribution shall be in compliance with the stress ranges of the S-N curve which forms basis for the assessment of the fatigue damage.

4.11.3.15 At welds, where stress singularities are present and extrapolation shall be applied to solve for the stress ranges, this implies that the same extrapolation procedure shall be applied to establish the stress ranges of the stress range distribution as the one that was used to establish the stress range values of the S-N curve for the weld.

Guidance note:

S-N curves are based on fatigue tests of representative steel specimens. During testing, stresses are measured by means of strain gauges. Stresses in the notch zone at the weld root and the weld toe cannot be measured directly, because strain gauges cannot be fitted sufficiently reliably at the notch due to the presence of the weld. Moreover, a stress singularity will be present in this area, i.e. stresses will approach infinity.

The stress which is recorded in standard fatigue tests is the so-called hot spot stress which is an imaginary reference stress. Hot spot stresses at weld toes and weld roots are established by extrapolation from stresses measured away from the notch zone. During testing for establishment and interpretation of an S-N curve, strain gauges are located in specific positions on the test specimens, and the hot spot stress is established by processing the measurements. To ensure an unambiguous stress reference for welded structural details, the strain gauge positions to be used for application of the strain gauges and for subsequent stress extrapolation are prescribed for each type of structural detail.

To fulfil the compatibility requirement, the stresses in the welds from the applied loading shall be determined as hot spot stresses for the weld in question, i.e. the stresses in the hot spots of the welds shall be computed by extrapolation from stresses in the extrapolation points defined by the laboratory tests for the relevant structural detail under consideration. Thus when a finite element analysis is used to determine the stresses in the welds due to the applied loading, the stresses in the welds shall be found by extrapolation of the stresses that are calculated by the analysis in extrapolation points in the very locations defined by the lab tests that have formed basis for the determination of the SN-curve.

Reference is made to DNVGL-RP-C203 for details regarding definitions of hot spots and principles for finite element modelling and analysis, including extrapolation of stresses to hot spots.

In case another design standard than DNVGL-RP-C203 is applied, such as EN 1993-1-9, the detail categories for hot spot stresses given in that standard should be applied together with the relevant tolerance requirements specified in that same standard.

For more complex methods to evaluate the FLS design such as the effective notch stress procedure, additional guidance is provided DNVGL-RP C203 App.E as well as in the IIW document IIW-1823-07 ex XIII-2151r4-07/XV-1254r4-07 *Recommendations for Fatigue Design of Welded Joints and Components*.

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

4.11.4 Characteristic cumulative damage and design cumulative damage

4.11.4.1 Predictions of fatigue life may be based on calculations of cumulative fatigue damage under the assumption of linearly cumulative damage, for example calculations according to Miner's rule. The characteristic stress range history to be used for this purpose can be based on rain-flow counting of stress cycles. The corresponding characteristic cumulative damage caused by this stress range history is denoted D_c .

Guidance note:

When Miner's rule is used for prediction of linear cumulative damage, the characteristic cumulative damage D_c is calculated as:

$$D_c = \sum_{i=1}^I \frac{n_{c,i}}{N_{c,i}}$$

where:

D_c is the characteristic cumulative damage,

I denotes the number of stress range blocks in a sufficiently fine, chosen discretisation of the stress range axis,

$n_{c,i}$ denotes the number of stress cycles in the i^{th} stress block, interpreted from the characteristic long-term distribution of stress ranges, and

$N_{c,i}$ is the number of repetitions of the cycle in the i^{th} stress block, which leads to failure, as interpreted from the characteristic S-N curve.

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

4.11.4.2 The design cumulative damage D_d is obtained by multiplying the characteristic cumulative damage D_c by the design fatigue factor DFF

$$D_d = \text{DFF} \cdot D_c$$

Design fatigue factors are specified in [4.11.5].

4.11.5 Design fatigue factors

4.11.5.1 The design fatigue factor DFF is a safety factor to be applied to the characteristic cumulative fatigue damage D_c in order to obtain the design fatigue damage.

Guidance note:

Because fatigue life is inversely proportional to fatigue damage, the design fatigue factor may be applied as a divisor on the calculated characteristic fatigue life to obtain the calculated design fatigue life.

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

4.11.5.2 The design fatigue factors in Table 4-18 apply to structures or structural components with low consequence of failure and for which it can be demonstrated that the structure satisfies the requirement for structural integrity in the ALS post-damaged condition with failure in the actual joint as the specified damage. The minimum design fatigue factors in Table 4-18 depend on the location of the structural detail and of the accessibility for inspection and repair. The relation between the level of inspection and the requirement for DFF is detailed in Sec.9. All surfaces designed to be inspection-free, regardless of whether they are accessible for inspection and repair, shall be treated as inaccessible in design. The requirement for $\text{DFF} = 3.0$ reflects an assumption of no or irregular inspections, regardless of accessibility. Lower DFFs may be used in the atmospheric and submerged zones if an inspection plan is established with documented inspection method and inspection intervals that will result in the same safety level as $\text{DFF} = 3.0$ without inspections.

Table 4-18 Required S-N curves and design fatigue factors - DFF

Location	Accessibility for inspection and repair of initial fatigue and coating damages ²⁾	S-N curve ⁵⁾	Minimum DFF ⁶⁾
Atmospheric zone	No	In air for coated surfaces free corrosion for surfaces protected by corrosion allowance, only ⁴⁾	3
	Yes		1
Upper splash zone (above MWL) ¹⁾	No	Combination of in air and free corrosion curves ^{3) 4)}	3
	Yes		2
Lower splash zone (below MWL) ¹⁾	No	In seawater for surfaces with cathodic protection	3
	Yes	Free corrosion for surfaces protected by corrosion allowance, only ⁴⁾	2
Submerged zone	No	In seawater for surfaces with cathodic protection	3
	Yes		Free corrosion for surfaces protected by corrosion allowance, only ⁴⁾
Scour zone	No	In seawater	3
Below scour zone	No	In seawater	3

Note:

- 1) Splash zone definition according to DNVGL-RP-0416.
- 2) If the designer considers the steel surface accessible for inspection and repair of initial fatigue damage and coating, this shall be documented through qualified procedures for these activities. See also [4.16] and Sec.9.
- 3) The basic S-N curve for unprotected steel in the splash zone is the curve marked free corrosion. The basic S-N curve for coated steel is the curve marked in air. It is acceptable to carry out fatigue life calculations in the splash zone based on accumulated damage for steel considering the probable coating conditions throughout the design life – intact, damaged and repaired. The coating conditions shall refer to an inspection and repair plan as specified in Sec.9.
- 4) When free corrosion S-N curves are applied in design, the full benefit of potential grinding of welds as outlined in [4.13.5] cannot be expected and therefore may not be taken into account. The effect of free corrosion on a ground weld may be accounted for by downgrading the S-N curve one class and applying the S-N curves for in seawater for free corrosion.
- 5) Shear keys within grouted connections may be designed assuming S-N curves marked in air.
- 6) According to the chosen DFF, an inspection program according to [9.3] will be required.

4.11.5.3 System effects are present wherever many weld segments in a long weld are subject to the same loading condition, such that the potential fatigue failure in the weld will take place in the weakest segment along the weld, i.e. the segment with the lowest fatigue strength. For welds where system effects are present, the system effects shall be accounted for in design, e.g. by appropriately increasing the DFF relative to the values specified in Table 4-18. For further details, reference is made to DNVGL-RP-C203 App.D.

4.11.6 Design cumulative fatigue by material factor approach

4.11.6.1 Present section covers an alternative to the DFF approach described in [4.11.4] and [4.11.5].

4.11.6.2 Design cumulative fatigue damage D_d can be calculated by Miner's sum as:

$$D_d = \sum_{i=1}^I \frac{n_{c,i}}{N_{d,i}}$$

where:

D_d is the design cumulative damage

I denotes the number of stress range blocks in a sufficiently fine, chosen discretisation of the stress range axis

$n_{c,i}$ denotes the number of stress cycles in the i^{th} stress block, interpreted from the characteristic long-term distribution of stress ranges, and

$N_{d,i}$ is the number of repetitions of the design cycle in the i^{th} stress block, $\Delta\sigma_{d,i} = \gamma_m \cdot \Delta\sigma_i$, which leads to failure, as interpreted from the characteristic S-N curve

γ_m is a material factor for fatigue

$\Delta\sigma_i$ is the stress cycle in the i^{th} stress block in the characteristic long-term distribution of stress ranges

4.11.6.3 The material factor γ_m for use with the method specified in [4.11.6] is a partial safety factor to be applied to all stress ranges before calculation of the corresponding numbers of cycles to failure that are used to compute the design fatigue damage.

4.11.6.4 The material factors in Table 4-19 are given as a function of the corresponding design fatigue factor DFF from Table 4-18 and are valid for structures or structural components, where the applied stress cycles during the design life are all in the $m = 5$ regime of an S-N curve. It shall be noted that this is likely not the case for stress cycles associated to pile driving.

Table 4-19 Material factors - γ_m

DFF	γ_m
1	1.0
2	1.15
3	1.25

4.11.7 Design requirement

4.11.7.1 The design criterion is:

$$D_d \leq 1.0$$

4.12 Fatigue limit states – bolted connections

4.12.1 Scope

4.12.1.1 Along with the general requirements stated in [4.11] for design against fatigue failure, present section summarises the design rules specifically pertaining to pre-loaded bolted connections subject to axial loads, shear loads or combinations thereof in the FLS.

4.12.2 Basic requirements

4.12.2.1 Basic requirements to bolt materials, geometry of bolt holes and preloading of connections are stated in [4.2.6] and [4.9.1] through [4.9.3].

4.12.3 Axially loaded connections

4.12.3.1 Stress cycles in bolts and surrounding base materials in axially loaded connections shall be determined based on an elastic model reflecting the axial stiffness of the two components. The preloading force used in the design calculations shall be guaranteed in the full lifetime of the support structure.

4.12.3.2 Potential loss of pre-tension shall be considered during design calculation. Consequently, the preload bolt force shall be used in the design calculations as:

$$F_{p'} = L_f \cdot F_p$$

4.12.3.3 L_f may be assumed to be a value of 0.90, if during the six months after first installation, but not directly after the installation, the pre-tension is ensured by retightening of the bolts if necessary.

4.12.3.4 Prying of bolt forces due to deformation of flange plates shall be accounted for.

Guidance note:

Note that compliance with the requirements stated in [4.9.4.4] (and pertaining to the connections' response under extreme external loads) implicates that in a linear elastic model the stress cycles in both bolts and surrounding base materials in the FLS will be simply proportional to the external cycles of loads.

For bolted flange connections in e.g. tubular towers and monopiles in which the individual bolted connections are predominantly axially loaded, the methods of analysis of bolts and flanges, which are described in Schmidt, H, Neuper, M. *Zum elastostatischen Tragverhalten exzentrisch gezogener L-Stöße mit vorgespannten Schrauben /3/*, may be applied for the computation of the stress cycles in the two components.

Computation of the stress cycles in the bolt and flange components may also be based on FE-models, adequately representing the real geometry of the connection including admissible imperfections and allowable tolerances.

Adequate compressive pre-loading of the flange contact surfaces may be assumed if the flatness deviation per flange (similar to $k/2$ in Figure 4-2) is within the tolerance value of 2 mm over the whole circumference and a maximum of 1 mm over an angle of up to 30° after completion of the manufacturing process.

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

4.12.3.5 The stress cycles in bolts and anchor rods shall always be referred to the tensile stress area, A_s .

4.12.3.6 The S-N curve for the bolts shall be taken according to DNVGL-RP-C203 [A.2] or EN 1993-1-9 (Table 8.1) depending on the selected basic design code and as relevant for the type of threading (rolling, cutting) and corrosion environment. See also [4.11.2.4].

Guidance note:

When referring to EN 1993-1-9 the following applies for axially loaded bolts of dimension up to and including 30 mm:

When using methods of analysis, which do not consider the influence of bending in the bolt, the detail category of the bolt should be taken as 36* (similar to Detail Category 40, but with knee point at $N_i = 1 \cdot 10^7$).

If the influence of bending of the bolt is directly accounted for in the analysis, higher detail categories may be referred according to the following rules:

- Bolt threads rolled before or after heat treatment, followed by thermal coating process (e.g. HDG): Detail category 50
- Bolt threads rolled before heat treatment, not followed by thermal coating process: Detail category 71
- Bolt threads rolled after heat treatment, not followed by thermal coating process: Detail category 71* $\cdot \left(2 - \frac{F_{S\max}}{F_{0,2\min}} \right)$, however not higher than 85 (71* :similar to Detail Category 80, but with knee point at $N_i = 1 \cdot 10^7$)

Here $F_{S\max}$ designates the maximum bolt force under extreme load including prestress and $F_{0,2\min}$ the bolt force at the 0,2% elastic strain limit.

For bolts of dimensions higher than 30 mm the detail categories should be as stated above reduced by the size effect factor

$$k_s = \sqrt[4]{\frac{30}{d}}$$

where d is the nominal diameter of the bolt [mm].

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

4.12.4 Shear loaded connections

4.12.4.1 Shear loaded bolted connections subject to fatigue shall be designed with standard or oversized round holes in all plates taking part in the connection. Slotted holes are not allowed in the design of these connections. See also [4.9.2].

4.12.4.2 End, edge and mutual distances between bolts in a shear loaded connection, which is subject to fatigue, shall be designed with minimum values as stated in Table 4-20.

Table 4-20 Minimum end, edge and mutual distances for bolts in shear loaded connections subject to fatigue

<i>Distance measure</i>	<i>Minimum value</i>
end distance (in direction of load)	$e_{1,\min} = 1.5 \cdot d_0$
edge distance (transverse to load)	$e_{2,\min} = 1.5 \cdot d_0$
pitch in direction of load	$p_{1,\min} = 2.5 \cdot d_0$
pitch transverse to load	$p_{2,\min} = 2.5 \cdot d_0$

4.12.4.3 Bolted shear loaded connections in wind turbine support structures, which are subject to dynamic loads, shall be designed so that the loading will not cause any local plate bending. Moreover, so that second-order axial stress cycles in the bolts will not be associated with the intended friction type load transfer.

Guidance note:

Compliance with this condition as well as the requirements stated in [4.9.5.2] (which pertain to the connections slip resistance under extreme external loads) implicates that in subject bolted connections there will be no nominal stress cycles in the bolts of connections subject to pure shear.

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

4.12.4.4 Regarding coefficients of friction and their dependence on surface treatments see [4.9.5].

4.12.4.5 The stress cycles in the plates of a shear loaded connection shall be determined based on an elastic model.

Guidance note:

For preloaded slip resistant shear connections the stress cycles in the plates may be simply computed based on the gross section of the plate (i.e. by disregarding the reduction of the section area due to the bolt holes and, also, by disregarding effects of stress concentration at the rim of the holes), provided they are held against an appropriately selected S-N curve and that all design rules regarding edge, end and mutual distances given in Table 4-20 above are complied with; see also [4.12.4.6] below.

Computation of the stress cycles in the plate components of the joined parts may also be based on FE-models, adequately representing the real geometry of the connection.

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

4.12.4.6 The S-N curve for the plates shall be taken according to DNVGL-RP-C203 [A.2] or EN 1993-1-9 Table 8.1 depending on the selected fundamental design code and as relevant for the joint design and corrosion environment. See also [4.11.2.4].

4.12.5 Connections subject to combined axial and shear loads

4.12.5.1 In general, bolted connections, which are subject to combined axial and shear loads in the FLS, shall be designed and a preloading of them also be specified so that they will be slip-resistant to all loads in the spectrum of operational loads.

Specifically, the following inequality applies in order for this requirement of slip resistance to be complied with:

$$F_{s,\max} \leq \frac{k_s n \mu F_{c,\min}}{1.10}$$

Here, $F_{s,\max}$ and $F_{c,\min}$ designate the characteristic maximum and minimum values of the shear and compressive loads transferred between the plates connected, the two being associated parts of same dynamic response, yet regardless of their individual variations in time. See also [4.9.5.5] for the definition and relevant values of k_s , n and μ .

4.12.5.2 For subject bolted connections, which comply with paragraph [4.12.5.1], the fatigue accumulation in bolts and plates can be assessed separately, i.e. as for the simple and 'purely' loaded connections dealt with in [4.12.3] and [4.12.4].

4.13 Fatigue limit states – welded connections

4.13.1 Scope

4.13.1.1 Present section specifies some basic design rules as well as specific methods for analysis of typical and rather frequently seen weld details in wind turbine foundations.

4.13.2 Tubular girth welds

4.13.2.1 Tubular girth welds shall be considered with appropriate detail categories according to DNVGL-RP-C203, or the project specific governing fatigue design standard.

4.13.2.2 A thickness transition located next to a girth weld will in combination with local offsets due to fabrication tolerances implicate a local bending and associated increase of stresses. These stresses shall be accounted for by an additional SCF multiplied to the nominal (far field) stress range.

4.13.2.3 SCF's applying to tubular girth welds may be calculated using analytical formulas, or by use of FE-models. In both cases, reference is made to DNVGL-RP-C203.

4.13.2.4 The distance between consecutive girth welds may not be less than

$$\Delta_{\min} = \begin{cases} 300\text{mm} & \text{for } D_o \leq 300 \text{ mm} \\ D_o & \text{for } 300 \text{ mm} < D_o < 1500 \text{ mm} \\ 1500\text{mm} & \text{for } D_o \geq 1500 \text{ mm} \end{cases}$$

where D_o designates the outer diameter of the tubular's section.

4.13.2.5 The minimum distance between attachment weld and girth weld shall be 300 mm.

Guidance note:

The minimum weld distances mentioned above have been derived based on practical experience. Shorter distances may be suitable, but need to be proven both with respect to impact on SCFs as well as residual stresses.

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

4.13.2.6 Improvement of fatigue properties for a girth weld may be considered according to clause [4.13.5].

Guidance note:

In general fatigue life improvement techniques should be applied with caution. DNV GL does not recommend the specification of fatigue life improvement techniques as a tool for the design process, as they should be reserved for uncertainties during fabrication and as a repair tool in the in-service phase.

It must be noted that benefit from fatigue life improvement techniques cannot be relied on in case of a free corrosion environment. Therefore, the D-curve according to DNVGL-RP-C203 should be considered the fundamental S-N curve to apply in a free corrosion environment.

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

4.13.3 Conical transitions

4.13.3.1 Conical transitions result in increased stresses in the shell wall due to local bending. These stresses shall be accounted for by an additional SCF multiplied to the nominal (far field) stress range.

4.13.3.2 SCF's at conical transitions may be calculated by use of analytical formulas or FE-models. In both cases, reference is made to DNVGL-RP-C203.

4.13.3.3 In case a conical transition is combined with a thickness transition, both effects shall be considered in the applied SCF.

4.13.4 Tubular joints

4.13.4.1 In tubular joints, local stress peaks shall be accounted for by applying appropriate SCFs. The SCFs shall consider both in-plane and out-of-plane bending.

4.13.4.2 SCFs for simple tubular joints may be calculated using Efthymiou's equations, see DNVGL-RP-C203.

4.13.4.3 SCFs for complex tubular joints may be calculated using FE-models.

4.13.4.4 Special requirements for fatigue assessment of roots of single side welded joints shall be considered, see DNVGL-RP-C203 [D.10].

4.13.4.5 Cans and stubs in tubular joints shall be designed with minimum lengths as specified in NORSOK N004 Section 6.4.

4.13.5 Improvement of fatigue performance of welds

4.13.5.1 The fatigue performance of welds in tubular joints or attachments can be improved by grinding. If the critical hotspot is at the weld toe, reduction of the local notch stresses by grinding the weld toe to a circular profile will improve the fatigue performance, as the grinding removes defects and some of the notch stresses at the weld toe.

4.13.5.2 If the grinding is performed in accordance with Figure 4-5, an improvement in fatigue life by a factor of 3.5 can be obtained. Further, the scale exponent, k , in the S-N curves may be applied according to DNVGL-RP-C203 App.D.

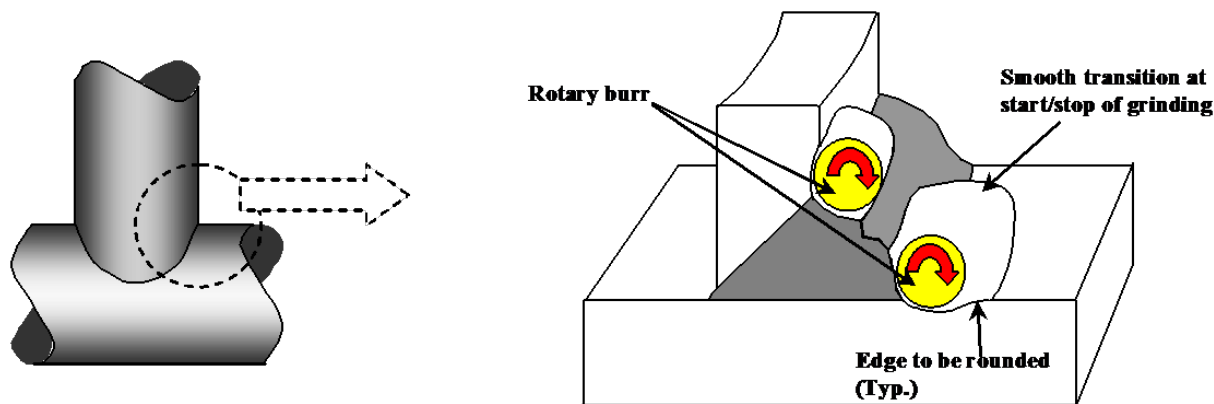


Figure 4-5 Weld toe grinding

4.13.5.3 The following conditions shall be fulfilled when welds in tubular joints are grinded:

- A rotary burr shall be used for grinding.
- Final grid marks should be kept small and always be normal to the weld toe, if the main loading is normal to the weld toe.
- The diameter of the burr shall be between 8 and 10 mm. If the brace plate thickness is less than 16 mm, the diameter of the burr may be reduced to 6 mm.
- The edges between the grinded profile and the brace/chord shall be rounded, i.e. no sharp edges are allowed.

- If the weld toe grinding shall not be performed on the complete circumference of the joint, a smooth transition between the grinded profile and the non-grinded weld shall be ensured.
- The grinded surface shall be proven free of defects by an approved non-destructive testing (NDT) method, e.g. MPI.
- The depth of grinding shall be 0.5 mm below any visible undercut. However, the grinding depth is normally not to exceed 2 mm or 5% of wall thickness whichever is less.

4.13.5.4 The fatigue performance of girth welds can be improved by grinding. Grinding of girth welds will increase the fatigue life of the welded connection if performed according to the conditions specified in [Figure 4-6](#).

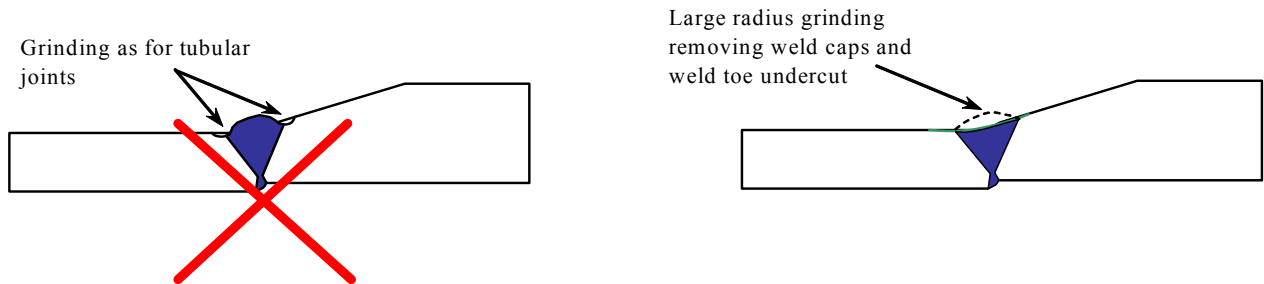


Figure 4-6 Girth weld grinding

4.13.5.5 When grinding of girth welds is carried out, local grinding by small-scale rotary burr (left) should not be performed. The preference is to perform profile grinding of the weld either of the weld cap alone as shown (right) or of the weld cap as well as the weld root.

4.13.5.6 If the weld is accomplished as a high-quality automated machine weld and the grinding is performed as shown to the right in [Figure 4-6](#) while also the below conditions are fulfilled, an improved S-N curve may be applied for the weld toe. If the weld root is grinded according to the same principles, an improved S-N curve may also be applied for the weld root. The SCF due to fabrication tolerances and geometry such as tapering shall still be applied, see also DNVGL-RP-C203.

- Final grid marks should be kept small and should always be normal to the weld toe.
- The largest radius possible considering the actual geometry shall be selected.
- The edges between the grinded profile and the brace/chord shall be rounded, i.e. no sharp edges are allowed.
- If the weld toe grinding shall not be performed on the complete circumference of the girth weld, a smooth transition between the grinded profile and the non-grinded weld shall be ensured.
- The grinded surface shall be proven free of defects by an approved NDT method, e.g. MPI.
- The depth of grinding shall be 0.5 mm below any visible undercut. However, the grinding depth is not to exceed 2 mm or 5% of wall thickness whichever is less.

4.13.5.7 Alternative methods of weld dressing as means for improvement of fatigue performance, such as profiling (flush grinding in the case of exterior faces of monopile girths) and TIG dressing, are described in DNVGL-RP-C203.

4.13.5.8 Fatigue life improvement achieved by way of dressing can be assessed according to DNVGL-RP-C203.

4.14 Accidental limit states design

4.14.1 General

4.14.1.1 The requirements in [4.14] regard the capability of the wind turbine steel support structure to withstand accidental events. They apply to the compound support structure as well as its structural members and relevant appurtenances.

4.14.1.2 In consideration of any one of these types of accidents, a design situation shall be defined, see for example [3.9]. Since the accidental events as such shall be considered mutually independent the design situation shall be defined as one dominated by the accidental event in question combined with expected and specified concurrent operating and environmental conditions.

4.14.1.3 The design basis shall also define the conditions relevant for ensuring the safety of the structures in relevant post damage situations where the structural system has been changed by the very accident and the originally designed configuration and functionality has not yet been restored.

These situations, which shall also be considered cases in the accidental limit states, shall include:

- expected gravity loads (G)
- specified variable loads (Q)
- specified environmental loads (E)
- but no accidental actions (A).

4.14.1.4 In lieu of more specific requirements, the environmental loads may be specified as aero- and hydrodynamic loads of a recurrence period not shorter than twice a conservative estimate of the time required to design, fabricate, inspect and install all repairs necessary to restore the structure's design resistance, however not less than one year.

4.14.1.5 In the analysis of the accidental limit states characteristic values of all loads and all resistance parameters shall be assumed.

4.14.2 Design requirements

4.14.2.1 The compound wind turbine support structure as well as its structural members and appurtenances shall be designed for the accidental scenarios defined for the specific support structure as well as for the situations defined for the structures in their post damage states.

4.14.2.2 Generally, when subject to the accidental event the structures may sustain damage in the appearance of large permanent deformations, loss of stability or disconnections, e.g. denting of a monopile shell wall, buckling failure of a brace in a jacket structure, formation of a plastic hinge in a boat bumper or rupture of a bolted joint.

4.14.2.3 Still, the integrated wind turbine support structure as well as its structural members and appurtenances shall be designed so that they will be able to withstand any of the hazards without completely losing their integrity or performance in consequence of the accidental events.

4.14.2.4 Regardless of the accident affecting the compound turbine foundation or any of its structural members and furnishings, the event itself shall be analysed by means of a model, which can reliably predict the structures' response, in particular its possible damages.

4.14.2.5 In some cases, this implicates the need for application of a non-linear structural analysis method.

4.14.2.6 The response of the damaged structure to the conditions defined for the post damage state may not allow for any progressive development of damages.

4.14.2.7 Acceptance criteria for damages to steel support structures caused by accidental events as well as requirements to their functionality in subsequent damaged conditions should be stated in the design basis. In lieu of such acceptance criteria the principles and guidelines stated below may be applied.

4.14.2.8 The design should ensure that after an accident affecting the compound wind turbine support structure, access will be possible under non-extreme conditions through a realistically estimated duration of evacuation of personnel (in case the structure is manned at the time of the accidental event) and salvage of valuable equipment.

Preferably, the design should allow for a reliable continuation of the use of the support structure under the post damage conditions specified in the design basis.

4.14.2.9 The structural members and appurtenances of the wind turbine support structure should be so designed that in the event of an accident directly affecting them, although possibly sustaining damages their continued use will be possible under the post damage conditions specified in the design basis.

4.14.2.10 In cases of impact (e.g. dropped objects), the accidental event shall be considered one of the structure absorbing all or part of the kinetic energy of the impacting object.

Ideally, at least in the case of the main structure being subject to an impact, the event should be analysed based on a dynamic time simulation of both action and response, the simulation capable of reflecting all phases of the impact as such and the process of the impact energy's dissipation.

The impacting object may be assumed to also take part in the energy absorption, typically by sustaining local denting or other permanent deformations.

As for the impacted structure, the analysis model for the energy absorption of the impacting object shall be fully documented if accounted for.

4.14.2.11 In cases of accidental actions (earthquake, hurricane, and freak wave) the action shall be taken as the best estimate of applied dynamic displacements (earthquake) or loads (hurricane, freak wave).

4.14.2.12 Ideally at least in the case of the main structure being subject to an accidental action, the event should be analysed based on a dynamic time simulation of both action and response.

Guidance note:

The structural analysis may most adequately be based on time series of accidental actions recorded during previous events in zones of natural conditions similar to the actual.

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

4.14.2.13 As regards cases of explosions or fire, the final and operating installation should be planned for so as to minimize their consequences for the structures. This could be achieved by avoiding confinements of potentially explosive matters and/or by planning and designing for installation of pressure relieving valves, fire suppressing equipment, etc., all as deemed appropriate.

4.14.2.14 More detailed methods of analysis of the accidental limit states are also advised in DNVGL-RP-C204.

4.14.2.15 It shall be noted that for wind turbine support structures the impact from a drifting services vessel is considered an abnormal load as described in DNVGL-ST-0437 [4.2.10] and [4.5.8]. Accidental service vessel impact is therefore handled as ULS.

4.15 Serviceability limit states design

4.15.1 General

4.15.1.1 The requirements in [4.15] regard the characteristic response of the wind turbine support structure and apply to the integrated structure as well as its structural members and equipment.

4.15.1.2 In the analysis of the serviceability limit states characteristic values of all loads and all resistance parameters shall be assumed.

4.15.1.3 Serviceability loads and requirements are defined in [3.10].

4.15.2 Member sections and joints

4.15.2.1 The structure shall be designed so that its response to the characteristic extreme load does not implicate initiation of yielding in any of its parts.

4.15.2.2 All bolted connections should be designed so that their characteristic ultimate capacity is not lower than the characteristic ultimate capacity of the members joined.

4.15.2.3 Axially loaded bolted connections (flange connections) shall be designed so that the characteristic extreme load will not cause yielding or loss of pre-tension. See also [4.9.4].

4.16 Corrosion protection

4.16.1 General

4.16.1.1 The requirements and guidance for corrosion control of wind turbine structures are in general given DNVGL-RP-0416 *Corrosion protection of wind turbines*. This recommended practice shall be followed, unless it can be demonstrated that an equivalent level of safety is met by other means.

4.16.2 Cathodic protection

4.16.2.1 For structures for which a CP design is applied, a CP system should be in place and operating as soon as possible after the installation of the structure. After maximum 365 days and minimum 30 days a CP survey shall be performed to confirm that the structures for which the cathodic protection design has been applied are adequately protected. In wind farms with many series manufactured structures with identical CP systems, it suffices to carry out the CP survey on a few representative structures only. In case of ICCP systems, rectifier parameters shall be checked for all structures and a potential survey shall be carried out if any inconsistency is established.

4.17 Fabrication and installation

4.17.1 General requirements and guidance for manufacturing

4.17.1.1 If the structure is designed and analysed according to the DNV GL series of standards the production of the support structure shall comply with DNVGL-OS-C401.

4.17.1.2 If the structure is designed and analysed according to EN 1993-1-1 the production of the support structure shall comply with EN 1090-2.

4.17.2 Structural category

4.17.2.1 The structural category shall be determined according to [4.2.3].

4.17.3 Welding procedures

4.17.3.1 If the structure is designed and analysed according to the DNV GL series of standards welding procedure specifications (WPS) and welding procedure qualification records (WPQR) shall comply with DNVGL-OS-C401 Ch.2 Sec.2.

4.17.3.2 If the structure is designed and analysed according to EN 1993-1-1 welding procedure specifications and welding procedure qualification records shall comply with the requirements according to EN 1090-2 Tables 12 and 13.

Guidance note:

For secondary structure components a welding procedure qualification according to EN ISO 15614-1 or EN ISO 15613 should be performed if the design verification is according to EN 1993-1-1.

Moreover, a test programme (test samples) which covers the various joint configurations, tube diameters, wall thicknesses and angles included between the axes of the tubes should be carried out. For the different welding positions for each individual connection type, the welding ability should be proven.

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4.17.4 Qualification of welders and welding supervisor

4.17.4.1 If the structure is designed and analysed according to the DNV GL series of standards the qualification of the welders shall comply with DNVGL-OS-C401 Ch.2 Sec.2 and Sec.3.

4.17.4.2 If the structure is designed and analysed according to EN 1993-1-1 the qualification of the welders shall comply with EN ISO 9606-1 and the qualification of the operator shall comply with EN ISO 14712. Under consideration of the respective execution class the qualification of the welding supervisor shall comply with the requirements in EN 1090-2 Table A.3.

4.17.5 Fabrication and tolerances

4.17.5.1 Contractors involved in fabrication and welding of structural members shall have a documented and implemented quality system according to ISO 9001 and welding approval depending on the structural category of the component to be fabricated.

4.17.5.2 If the structure is designed and analysed according to EN 1993-1-1 the factory production control (FPC) has to be inspected within an initial audit and continuous monitoring, assessment and approval of FPC has to be performed.

Guidance note:

It is recommended to perform an initial audit in accordance with agreed inspection and test plans before the production starts to confirm that all project specific requirements are fulfilled.

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4.17.6 Material identification and traceability

4.17.6.1 Guidance for the material identification and traceability is given in DNVGL-OS-C401 Ch.2 Sec.6.

4.17.6.2 A description of the material identification system and the traceability system for certification of acceptance should be submitted prior to commencement of fabrication.

4.17.7 Tolerances and assembly

4.17.7.1 If the structure is designed and analysed according to the DNV GL series of standards the tolerances shall comply with the requirements in DNVGL-OS-C401 Ch.2 Sec.6.

4.17.7.2 If the structure is designed and analysed according to EN 1993-1-1 the tolerances shall comply with EN 1090-2 Appendix D. Additional requirements may be necessary due to the requirements related to the applied S-N Curve.

Guidance note:

Typical examples are e.g. misalignment of butt welds, out-of-roundness of tubular members, flange flatness tolerances.

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4.17.7.3 Weld clustering at tubular joints of offshore structures shall comply with ISO 19902 Section 20.4.2.2.

4.17.8 Non-destructive testing

4.17.8.1 Non-destructive testing (NDT) shall be performed in accordance with agreed written procedures. General requirements for non-destructive testing are given in DNVGL-OS-C401 Ch.2 Sec.7.

4.17.8.2 If the structure is designed and analysed according to the DNV GL series of standards the NDT procedures shall be in accordance with DNVGL-CG-0051.

4.17.8.3 If the structure is designed and analysed according to EN 1993-1-1 the NDT procedures shall comply with the standards given in EN 1090-2 Section 12.4.2.4.

Guidance note:

Testing of single sided welds at tubular joints should be performed in accordance with API RP 2X. An appropriate procedure should be provided for NDT of tubular joints for certification for acceptance.

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4.17.9 Personnel qualification for non-destructive testing

4.17.9.1 If the structure is designed and analysed according to the DNV GL series of standards the qualification for personnel performing NDT shall comply with the requirements stated in DNVGL-OS-C401 and DNVGL-CG-0051.

Guidance note:

A hierarchy of standards for the manufacturing requirements should be agreed at the beginning of the project. In case the wind turbine structures are manufactured in accordance with DNVGL-OS-C401 the requirements listed in DNVGL-OS-C401 Ch.3 with respect to qualification of companies may be excluded.

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4.17.9.2 If the structure is designed and analysed according to EN 1993-1-1 the qualification for personnel performing NDT shall comply with EN ISO 9712.

Guidance note:

It is recommended to qualify and test the NDT personnel for testing of single side welds for tubular joints, if the design verification is performed according to EN 1993-1-1. Guidance for test procedures are given in API RP 2X.

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4.17.10 Extent of non-destructive testing

4.17.10.1 If the structure is designed and analysed according to the DNV GL series of standards the scope of NDT shall comply with the requirements according to DNVGL-OS-C401 Ch.2 Sec.7. The inspection category is by default related to the structural category according to [Table 4-21](#).

Table 4-21 Inspection category

<i>Inspection category</i>	<i>Structural category (see Table 4-1)</i>
I	Special
II	Primary
III	Secondary

4.17.10.2 If the fabrication quality is assessed by testing, or if it is of a well-known quality based on previous experience, the extent of inspection may be lowered as shown in [Table 4-22](#).

4.17.10.3 If the structure is designed and analysed according to EN 1993-1-1 the scope of NDT shall comply with EN 1090-2 Table 24.

4.17.10.4 The weld connection between two components shall be assigned to the inspection category given as the highest category of the joined components.

4.17.10.5 Fatigue-critical details within structural category primary and secondary should in general be inspected according to a category that is one level higher. Deviations from this rule can be made as shown in [Table 4-22](#).

4.17.10.6 Welds between flanges and adjacent shells shall always be assigned to the inspection category I.

4.17.10.7 For monopile type structures, the longitudinal welds in the monopile and in the transition piece in the segments of the grouted connection shall be inspected according to requirements given for category I.

Table 4-22 Extent of inspection

<i>Structural category</i>	<i>Special</i>	<i>Primary</i>	<i>Secondary</i>
Non-fatigue critical details	I	II	III
Non-fatigue critical details, well proven track record for production (industrial production leading to a uniform and predictable quality, see also DNVGL-OS-C101)	II	III	III
Fatigue critical details	I	I	II
Fatigue critical details, well proven track record for production (industrial production leading to a uniform and predictable quality, see also DNVGL-OS-C101)	I	II ¹⁾	III

<i>Structural category</i>	<i>Special</i>	<i>Primary</i>	<i>Secondary</i>
<i>Notes:</i>			
1) Examples of fatigue critical details categorized as primary where well proven track record often exists are shell-shell connections for towers, MP's and TP's.			

Guidance note:

It is recommended to apply the scope of NDT according to DNVGL-OS-C401 if the design verification is performed according to EN 1993-1-1. Weld beads are classified as inspection category I – filled welds.

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4.17.11 Hold time for non-destructive testing

4.17.11.1 If the structure is designed and analysed according to the DNV GL series of standards the hold time after completion of the weld shall comply with the requirements according to DNVGL-OS-C401 Ch.2 Sec.7 [1.2.4].

4.17.11.2 If the structure is designed and analysed according to EN 1993-1-1 the hold time after completion of the weld shall comply with the requirements given in EN 1090-2 Table 23.

Guidance note:

It is recommended to apply a minimum hold time after completion of the weld of 24 h for steel grades with a specified minimum yield strength (SMYS) of 360 MPa or lower and a hold time of 48 h for steel grades with a SMYS of 420 MPa if the plate thickness is more than 40 mm in order to detect hydrogen induced delayed cracks.

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4.17.12 Acceptance criteria for non-destructive testing

4.17.12.1 If the structure is designed and analysed according to the DNV GL series of standards the acceptance criteria shall be in accordance with DNVGL-OS-C401 Ch.2 Sec.7 considering the respective structural category.

4.17.12.2 If the structure is designed and analysed according to EN 1993-1-1 the acceptance criteria shall be in accordance with EN 1090-2 Table A.3 considering the respective execution class.

4.17.12.3 Except for joints in monopiles, post weld heat treatment (PWHT) shall be applied for joints in steels with a SMYS larger than or equal to 420 MPa in special areas when the material thickness at the welds exceeds 50 mm. For details, see DNVGL-OS-C401 Ch.2 Sec.6. As an alternative to PWHT, CTOD test results for weld metal and HAZ shall be documented and meet a requirement of minimum 0.25 mm.

4.17.13 Installation

Re-tightening of bolts and anchor rods shall be ensured not later than six month after first installation but not directly after the installation.

Guidance note:

10 days (240 hours) of turbine operation after installation should be considered as a minimum.

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SECTION 5 CONCRETE STRUCTURES

5.1 Introduction

5.1.1 General

5.1.1.1 For wind turbine concrete structures a coherent set of standards for materials, design and construction shall be used and supplemented by the provisions in this section. Either DNVGL-ST-C502 or EN 1992-1-1 should be used as the basic design standard. The requirements in this section are based on the assumption that one of these standards forms the basis for the design. Alternatively, or as a supplement, other standards can be used as specified in [1.2.4]. It is the responsibility of the designer to document that the requirements in [1.2.4] are met.

5.1.1.2 The loads to be determined for design of a wind turbine concrete support structures are specified according to Sec.3. Details regarding the process of determining the load effects within the concrete structure can be found in DNVGL-ST-C502 or EN 1992-1-1 respectively.

5.1.1.3 This section in general provides requirements and guidance which are supplementary to the provisions of DNVGL-ST-C502 and EN 1992-1-1 for its application on wind turbine concrete structures. For all design purposes, the user should always refer to the complete description in DNVGL-ST-C502 or EN 1992-1-1 together with this section.

5.2 Concrete structure concepts

In this subsection typical concrete support structures are described. For geotechnical elements of the structures see Sec.7 of this standard.

5.2.1 Concrete towers

Concrete towers consist of reinforced concrete. For the cross-sections of the tower different shapes can be used (circle shape, squared shape, octagonal shape etc.). They can be made of in-situ cast concrete as well as precast elements. Concrete towers can be pre-tensioned or post-tensioned in vertical as well as in horizontal direction. For the connection to the nacelle a steel adapter/transition piece is typically used.

5.2.2 Hybrid steel-concrete towers

A hybrid steel-concrete tower is a combination of a reinforced concrete part used for the lower section and a steel part used for the upper section. For the concrete part the same concepts can be used as described in [5.2.1]. The steel part shall follow the principles of Sec.4 in this standard.

5.2.3 Onshore gravity-based foundations

Slab foundations are used for onshore wind turbines founded on firm soils. Different geometries can be used (e.g. circle shape, squared shape, polygonal shape). The connection between a slab foundation and the tower are typically designed with an anchor bolt connection or with embedded steel can section. Slab foundations are usually partly covered with soil. The dead load of the soil overburden can be considered for the verification of the stability of the foundation. The bearing resistance relies upon the deadload of concrete and soil overburden (hence gravity-based foundation).

5.2.4 Onshore pile foundations

Pile foundations are used in case that the upper soil layers do not have a good bearing capacity. Pile foundations consist of a pile cap which is connecting the piles. Different geometries can be used for the pile cap (e.g. circle shape, squared shape etc.). For the connection between tower and pile cap anchor bolts or embedded steel cans can be used. The loads are transmitted by a combination of skin friction and tip resistance of the piles.

5.2.5 Onshore rock anchor foundations

Rock anchor foundations consist of a concrete cap connected to steel bars or cables grouted into boreholes (rock anchors). The rock anchors are post-tensioned. Forces are transmitted to the rock by compression below the concrete cap as well as tension in the rock anchors. Rock anchor foundations can be used where the bedrock resistance is sufficient already at lower depth.

5.2.6 Offshore gravity-based structures

5.2.6.1 Offshore gravity-based structures are substructures held in place by gravity. They can be built using in-situ concrete as well as precast element solutions and are typically constructed onshore and towed to their final position offshore. Tanks or cells inside the gravity-based structure can be used to control buoyancy during transportation. By flooding the cells with water the gravity-based structure are set down. In order to have sufficient ballast, cells can be filled with sand, rock or other materials.

5.2.6.2 The structural behavior of an offshore gravity-based structure is typically similar to an onshore slab foundation, see [5.2.3].

5.2.6.3 Scour protection is required for this type of structure.

5.2.7 Offshore gravity-based pile structure

Offshore gravity-based pile structures are held in place by gravity and by piles. The concept is similar to the offshore gravity-based structures, see [5.2.6]. The bearing resistance of the whole structure is derived by a combination of gravity and pile resistance.

5.3 Materials

5.3.1 General

5.3.1.1 Reinforced concrete as well as prestressed concrete with normal or high strength may be used for support structures of wind turbines. The concrete can be used as precast concrete or in-situ cast concrete.

5.3.1.2 Materials selected for the load-bearing structures shall be suitable for the purpose. Material specifications shall be established for all materials used for manufacturing the concrete, for the reinforcement as well as for the pre-tensioning/post-tensioning system.

5.3.1.3 Selected materials shall comply with the basic design standard unless the requirements are amended in this standard.

5.3.1.4 If materials are used that are not mentioned explicitly in the applied design standard, it has to be verified that these material properties fulfill the requirements of the relevant standards.

5.3.2 Concrete/grout constituents

5.3.2.1 Constituent materials for structural concrete are cement, aggregates and water. Structural concrete and grout may also include admixtures and additions.

5.3.2.2 If the design verification is performed according to DNVGL-ST-C502, the constituents shall fulfill the requirements in DNVGL-ST-C502 Sec.4.

5.3.2.3 If the design verification is performed according to EN 1992-1-1, the constituents shall comply with EN 206.

Guidance note:

Low heat cement should be used where heat of hydration may have an adverse effect on the concrete during curing.

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5.3.2.4 Chlorides, chloride-bearing or other steel-corrosion-promoting materials shall not be added to reinforced or prestressed concrete.

5.3.3 Concrete

5.3.3.1 If the design verification is performed according to DNVGL-ST-C502, the concrete specifications shall be based on the requirements in DNVGL-ST-C502 Sec.4. Normal Strength Concrete according to DNVGL-ST-C502 is a concrete of grade C35 to C65 and High Strength Concrete according to DNVGL-ST-C502 is a concrete of grade in excess of C65.

5.3.3.2 If the design verification is performed according to EN 1992-1-1, the concrete specification and production shall be based on EN 206. Normal strength concrete according to EN1992-1-1 is a concrete of grade up to C50/60 and high strength concrete according to EN1992-1-1 is a concrete of grade C55/67 or higher.

5.3.3.3 The concrete composition and the constituent materials shall be selected to satisfy the requirements of the basic standards, requirements in this subsection, and the project specifications for fresh and hardened concrete such as consistency, density, strength, durability and protection of embedded steel against corrosion. Due account shall be taken of the methods of execution to be applied. The requirements of the fresh concrete shall ensure that the material is fully workable in all stages of its manufacture, transport, placing and compaction.

5.3.3.4 For onshore concrete structures the water/cement-ratio shall not be greater than 0.5. In case of highly aggressive chemical environment and for offshore concrete structures the water/cement-ratio shall not be greater than 0.45. In the splash zone of offshore concrete structures, this value shall not be higher than 0.40.

5.3.3.5 The minimum cement content shall meet the requirements according to the basic design standard for reinforced or prestressed concrete not within the splash zone. In the splash zone the minimum requirement to cement content is 375 kg/m^3 . Alternatively if it is proven during qualification testing that the concrete has robust durability performance a deviation may be made.

Guidance note:

Performance in respect to durability could be proven by testing in accordance with a recognised test standard such as BS 1881-122:2011 in order to prove water absorption of less than 11%. Other tests may be required if a design life outside the scope of this standard is considered.

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5.3.3.6 The total chloride ion content of the concrete in the splash zone shall not exceed 0.10% of the weight of cement in ordinary reinforced concrete and in concrete containing prestressing steel.

5.3.3.7 Concrete subjected to freezing and thawing is to have adequate frost resistance. For structures designed according to DNVGL-ST-C502 the minimum air content shall be specified according to DNVGL-ST-C502 Sec.4. If the design is performed according to EN 1992-1-1, it shall have a minimum air content of 4%.

5.3.4 Structural grout

5.3.4.1 The mix design of structural grout shall be specified for its designated purpose.

5.3.4.2 If the design verification is performed according to DNVGL-ST-C502, the grout specifications shall be based on the requirements in DNVGL-ST-C502 Sec.4.

5.3.4.3 If the design verification is performed according to EN 1992-1-1, the grout specification and production shall be based on EN 206.

5.3.4.4 Structural grout can be used for onshore (e.g. tower/foundation, joints of precast elements, etc.) as well as for offshore application (e.g. grouted connection, see [Sec.6](#)).

5.3.4.5 In general the behaviour of grout material can be compared to concrete. The strength of the (ultra-) high strength grout is considered out of the range of current standards for concrete.

5.3.4.6 The constituents of structural grout shall meet the same type of requirements for their properties as those given for the constituents of concrete.

5.3.4.7 For an evaluation of the suitability of the grout the environmental conditions for the designated area of application, especially for offshore conditions, need to be considered.

5.3.4.8 A grout certification consists of the assessment of the grout material with regard to design relevant material properties as well as a shop approval.

5.3.4.9 Expected ambient conditions during mixing, curing and hardening (e.g. min/max temperatures) shall also be considered as part of the grout certification. As a minimum the tests shall be conducted at a fresh concrete and test temperature of 20°C as well as at the lowest fresh concrete and ambient temperature at which the wet and hardened concrete properties are defined by the customer.

5.3.4.10 Material properties of fresh grout (FG) and hardened grout (HG) shall be verified based on acknowledged standards.

5.3.4.11 The scope of the required material tests for structural grout can be found in DNVGL-ST-C502 [H.2] *Testing of materials*. The following test methods according EN standards are also accepted for the grout certification as an alternative:

Table 5-1 Alternative standards for grout testing

<i>Test id</i>	<i>Type of test</i>	<i>Test method</i>
FG1	Flow Test	EN 12350-8
FG3	Bleeding / Segregation	EN 12350
HG4	Flexural strength	EN 12390-5
HG7	Static Young's Modulus & Poisson's ratio	EN 12390-13

5.3.4.12 The compressive strength of the grout is based on the characteristic uniaxial cylinder compressive strength f_{ck} measured on 150 × 300 mm cylinders after 28 days. For the continuous quality control other specimen geometries can be used, but the strength has to be modified by a geometry factor determined as part of the grout certification.

5.3.4.13 All required tests for the grout certification for the grout material shall be performed by an accredited laboratory or follow comparable procedures.

5.3.4.14 If the grout material is reinforced by fibre the required tests for the “grout certification” may deviate.

5.3.5 Reinforcement steel

5.3.5.1 Reinforcements shall be suitable for their intended service conditions and are to have adequate properties with respect to strength, ductility, toughness, weldability, bond properties (ribbed), corrosion resistance and chemical composition. These properties shall be specified by the supplier or determined by a recognised test method.

5.3.5.2 If the design verification is performed according to DNVGL-ST-C502, the reinforcement shall fulfill the requirements of DNVGL-ST-C502 Sec.4.

5.3.5.3 If the design verification is performed according to EN 1992-1-1, the reinforcement shall comply with EN 10080.

5.3.5.4 Fatigue properties and S-N curves shall be consistent with the assumptions of design.

5.3.6 Prestressing steel

5.3.6.1 If the design verification is performed according to DNVGL-ST-C502, the prestressing steel and the prestressing system shall fulfill the requirements of DNVGL-ST-C502 Sec.4.

5.3.6.2 If the design verification is performed according to EN 1992-1-1, the prestressing steel and the prestressing system shall comply with EN 10138 and shall have a valid European Technical Approval (ETA).

5.4 Structural design

5.4.1 Design material strength

5.4.1.1 In design by calculation according to this standard, the design material strength shall be determined by applying a material factor γ_m .

5.4.1.2 For design according to DNVGL-ST-C502 the material factors, γ_m , for concrete and reinforcement for wind turbine concrete structures are given in [Table 5-2](#).

Table 5-2 Material factors for concrete and reinforcement

Limit state		ULS	FLS	ALS	SLS
Reinforced concrete/grout ³⁾	γ_c	1.50 ¹⁾ (1.35) ²⁾	1.50 ¹⁾ (1.35) ²⁾	1.30 ¹⁾ (1.20) ²⁾	1.0
Steel reinforcement	γ_s	1.15 ¹⁾ (1.1) ²⁾	1.10 ¹⁾ (1.00) ²⁾	1.10 ¹⁾ (1.00) ²⁾	1.0

Limit state		ULS	FLS	ALS	SLS
Fibre reinforced concrete/grout	γ_c	1.50 ¹⁾ (1.35) ²⁾	1.50 ¹⁾ (1.35) ²⁾	1.30 ¹⁾ (1.20) ²⁾	1.0
Plain concrete/grout	γ_c	1.80	1.80	1.50	1.0

Notes:

- 1) Design with these material factors allows for tolerances in accordance with DNVGL-ST-C502 [6.3.6] or, alternatively, tolerances for cross-sectional dimensions and placing of reinforcements that do not reduce the calculated resistance by more than 10%. If the specified tolerances are in excess of those given in DNVGL-ST-C502 [6.3.6] or the specified tolerances lead to greater reductions in the calculated resistance than 10%, then the excess tolerance or the reduction in excess of 10% shall be accounted for in the resistance calculations. Alternatively, the material factors may be taken according to those given under 2).
- 2) When the design shall be based on dimensional data that include specified tolerances at their most unfavourable limits, structural imperfections, placement tolerances as to positioning of reinforcement, then these material factors may be used. When these factors are used, then any geometric deviations from the approved for construction drawings shall be evaluated and considered in relation to the tolerances used in the design calculations.
- 3) Material factors for reinforced grout may be used in design where the grout itself is reinforced by steel reinforcement or where it can be demonstrated that steel reinforcement or anchor bolts in the surrounding structure contributes to reinforce the grout (such as grouted connection type B in DNVGL-ST-C502 [6.20.7]).

5.4.1.3 In design by calculation according to DNVGL-ST-C502 the material factor γ_c for material strength of concrete shall be applied as a divisor on the normalized compressive strength f_{cn} and not as a divisor on the characteristic compressive strength defined as the 5% quantile in the probability distribution of the compressive strength of concrete (f_{cck}). The normalized compressive strength is calculated as: $f_{cn} = f_{cck} \cdot (1 - f_{cck}/600)$.

Further, for calculation according to DNVGL-ST-C502 for a wind turbine support structure design; $\alpha_c = 1.0$ and $\alpha_t = 1.0$ may be applied in all limit states, i.e. in both ULS, SLS, FLS and in ALS.

5.4.1.4 For design according to EN 1992-1-1 the material factors, γ_m , for concrete and reinforcement shall be taken directly according to EN 1992-1-1.

Guidance note:

In design by calculation according to EN 1992-1-1 the material factor γ_m for material strength of concrete shall be applied as a divisor on the product of f_{ck} and α_{cc} as specified in EN 1992-1-1 Section 3.1.6.

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5.4.2 Thermal actions

5.4.2.1 Heat influences on reinforced or prestressed concrete towers and other hollow cross sections shall be taken into account. For normal environmental conditions as specified in DNVGL-ST-0437 a temperature gradient of $\Delta T = 15K$ acting uniformly over the circumference and varying linearly through the wall thickness shall be combined with a temperature gradient of $\Delta T = 15K$ acting with a cosine distribution over a circumferential sector from 0° to 180° as shown on [Figure 5-1](#).

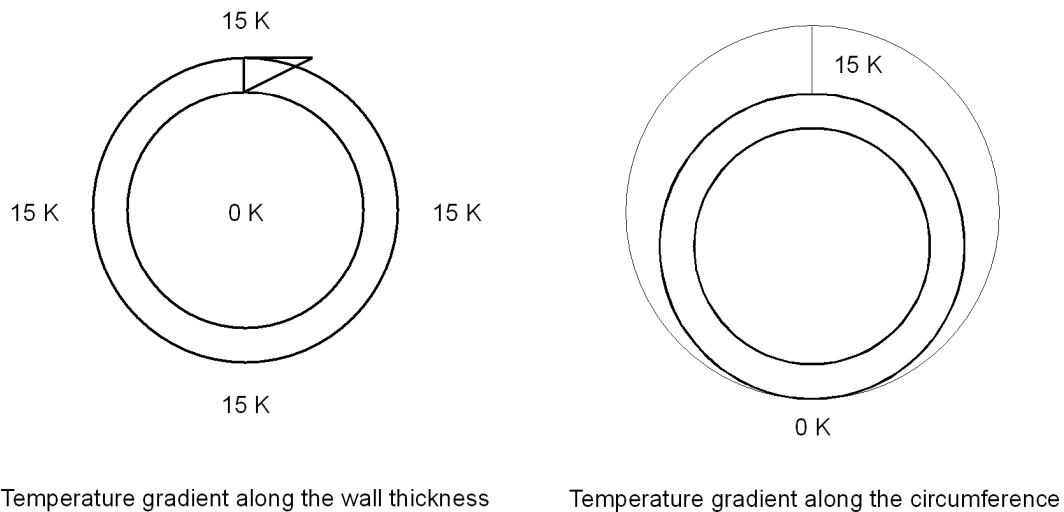


Figure 5-1 Temperature action on concrete towers

5.4.2.2 For ULS and SLS the temperature load combinations according to [5.4.2.1] have to be combined with the load assumptions according to Sec.3 and DNVGL-ST-0437. In this combination of actions a load combination factor of 0.6 may be applied.

5.4.3 Load transfer from steel tower to concrete foundation

5.4.3.1 The connection between a steel tower and a concrete foundation shall be designed with due consideration of the tower geometries and tolerances. As described in [5.2] such connections are typically designed with either cast in anchor bolts or with embedded steel can section.

Guidance note:

The design interfaces between the tower designer and the foundations designer should be clarified before the design work is initiated.

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5.4.3.2 The concrete foundation shall be designed in all limit states for local pressure loads from the steel elements including possible stresses from anchorage of post-tensioned anchor bolts.

5.4.3.3 Conventional design methods for such regions with discontinuity in geometry are typically not developed for the large dimensions which are often applicable for wind turbine foundations. Therefore, special care shall be taken when selecting design methods.

Guidance note:

If no suitable conventional methods are available it may be beneficial to apply different calculations methods to the load transfer detail in order to verify the robustness of the methods.

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5.4.4 Load transfer from concrete slab to stone bed

5.4.4.1 If offshore gravity-based structures shall be installed on a stone bed the local pressure loads on the concrete structure from the stones shall be considered. The local pressure loads will be related to for example stone size and tolerances on the installed stone bed.

5.4.5 Load-dependent stiffness reduction

5.4.5.1 For the calculation of bending moments along a concrete tower using second-order theory, load-dependent stiffness reduction shall be applied. The stiffness reduction due to the second-order effect itself can be considered negligible.

5.4.5.2 For the calculation specified in [5.4.5.1] the load factor relevant for the considered limit state shall be applied.

5.4.6 Dry joints

5.4.6.1 For designs with dry joints (i.e. direct contact between precast concrete elements) the tolerances of the jointed surfaces shall be controlled in order to avoid high stress concentrations. The specified tolerances shall be validated in the design by testing and/or FE analysis.

5.4.7 Stress distribution through concrete

5.4.7.1 The stresses underneath concentrated forces can in general be assumed to distribute through concrete and grout under an angle equal to $\arctan(1/2) = 26.6^\circ$ relative to the force direction. If a higher angle is assumed this shall be thoroughly evaluated.

5.4.8 Design assisted by testing

5.4.8.1 Model tests can be used to verify the strength and distribution of cross-sectional values if they are carried out by acknowledged institutions with the relevant experience.

5.4.8.2 The guidance in Annex K of IEC 61400-1 Ed. 4 should be taken into account.

5.5 Ultimate limit states

5.5.1 General

5.5.1.1 Analysis of components made of reinforced concrete or prestressed concrete shall be based on the concept of partial safety factors for design loads (see [Sec.3](#)).

5.5.1.2 If DNVGL-ST-C502 has been chosen as the basic design standard the ULS verification shall follow this standard.

5.5.1.3 In case of using EN 1992-1-1 as the basic design standard, the design shall be performed based on this standard.

5.5.1.4 In ULS the load effects can be determined according to either linear elastic analysis or plastic analysis as described in the basic design standard.

5.5.1.5 The maximum strain of the reinforcement steel in the cross-section should not exceed a limiting value of at 1.0%. Use of higher strains shall be justified and documented.

5.5.1.6 The increase in the internal forces and moments through non-linear influences (e.g. second-order theory, crack formation) shall be taken into account. It may be determined from a quasi-static calculation.

5.5.1.7 Structures in which the failure or defect of one component can lead to the collapse of further components should be avoided.

5.6 Fatigue limit states

5.6.1 General

5.6.1.1 For components of reinforced concrete or prestressed concrete, a detailed fatigue analysis shall be provided for the concrete, the reinforcing steel and the prestressing steel.

5.6.1.2 Load effects should in FLS be determined based on theory of elasticity. If this is not applied the validity of the chosen methods shall be documented.

5.6.1.3 The verification should be performed based on Markov matrices for all fatigue governed details.

5.6.2 Reinforcement and prestressing steel

5.6.2.1 The capacity to resist dynamic loading for reinforcement and prestressing steel shall be verified according to the selected basic design standard. If EN 1992-1-1 has been selected as the basis design standard also Model Code 2010 may be applied for design in the fatigue limit states.

5.6.2.2 The verification should preferably be performed by Markov matrices. If an equivalent fatigue damage load or a rain-flow-count spectrum is used for the verification it shall be shown that a linear relationship between loads and load effects can be assumed.

5.6.2.3 The cumulative damage can be calculated according to Palmgren Miner's rule.

5.6.2.4 If DNVGL-ST-C502 has been selected as the basic design standard, also the fatigue design shall follow this standard. If structures are not planned to be inspected by close visual inspections at regular intervals the cumulative fatigue damage shall be limited to 0.33. The cumulative fatigue damage shall for reinforcement and prestressing steel never exceed 0.5 as described in DNVGL-ST-C502.

5.6.2.5 If the fatigue analysis is made according to EN 1992-1-1 or Model Code 2010, the cumulative damage shall be below 1.0 at all location.

5.6.2.6 For concrete structures in exposure class XD1, XD2, XD3, XS1, XS3, XF2, XF3, XF4, and XA3 (as specified in EN 206) the method according to EN 1992-1-1 shall not be applied. Instead the method for marine environment according to Model Code 2010 may be applied.

5.6.2.7 It shall be shown that the reinforced concrete structure has redundancy in case of fatigue failure in one reinforcement bar or prestressing tendon. By redundancy is meant that there are multiple load paths i.e. that the structure can redistribute the loading and thereby not have a lower ultimate carrying capacity after failure in one bar or tendon. It is further a requirement that the stresses in none of the remaining intact bar or tendons will increase significantly.

Guidance note:

Redundancy may often be shown for thick concrete slabs where only limited plastic distribution of internal forces have been assumed in ULS design. By a significant increase is meant more than 5% increase of the stresses calculated for the highest fatigue loads.

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5.6.2.8 If redundancy cannot be shown for a structure where DNVGL-ST-C502 has been selected as the basic design standard the material factor for consideration of tolerances according to Note 2 in [Table 5-2](#) cannot be applied.

5.6.2.9 If the fatigue analysis is made according to EN 1992-1-1 or Model Code 2010 a material factor of $\gamma_s = 1.25$ shall be used if redundancy cannot be shown.

5.6.2.10 For reinforcement bent around a mandrel (for example shear reinforcement) it shall be evaluated whether the fatigue failure shall be expected at the straight part of the reinforcement bar. If not the reduced fatigue strength at the bent shall be taken into account according to the selected basic design standard.

5.6.3 Detailed fatigue verification for concrete

5.6.3.1 For areas of a concrete support structure where a simplified fatigue capacity check according to [\[5.6.4\]](#) is not met a detailed fatigue verification shall be made.

5.6.3.2 The verification shall be performed by means of detailed Markov matrices.

5.6.3.3 The cumulative damage can be calculated according to Palmgren Miner's rule.

5.6.3.4 For fatigue verification of concrete structures a distinction is made between in air conditions and in water conditions. In water conditions shall be considered for permanently submerged structures, for splash zones of offshore structures, for structures subjected to long-term water contact (for example from ground water) and for structural parts where water can assemble (i.e. many outdoor horizontal surfaces). Examples of exposure classes (as specified in EN 206) for which in water condition shall be considered are; XC2, XD2, XS2, and XS3. Examples for which in air condition can be considered are; X0, XC3, XC4, XD1, XS1, XF1, and XF2. For any other exposure classes an individual assessment has to be made.

In water conditions shall in most cases apply to grout cast underneath steel tower flanges, if this detail is not thoroughly protected from water ingress.

5.6.3.5 If DNVGL-ST-C502 has been selected as the basic design standard, also the fatigue design shall follow this standard. If structures (both onshore and offshore) or areas of structures are not planned to be inspected by close visual inspections at regular intervals the cumulative fatigue damage shall be limited to 0.33.

5.6.3.6 DNVGL-ST-C502 can be applied for detailed fatigue design for structure both in air and in water as according to the description in DNVGL-ST-C502. This applies for both onshore and offshore concrete structures.

5.6.3.7 If the structure is designed and analysed according to EN 1992-1-1 the detailed fatigue design shall be performed based on either EN 1992-2 or Model Code 2010. The cumulative damage determined from one of these methods shall be limited to 1.0 at all location of the structures.

5.6.3.8 For structural parts in water where EN 1992-2 or Model Code 2010 is applied for the detailed fatigue design the calculated allowable number of cycles (N_i) shall be reduced for each stress-block before the related part damage is calculated. For the stress-blocks having stress variations in the compression-compression range N_i shall be raised to the power of 0.8 (i.e. $N_i^{0.8}$) and for the stress-blocks having stress variations in the compression-tension range N_i shall be raised to the power of 0.65 (i.e. $N_i^{0.65}$).

5.6.3.9 For fatigue design according to EN 1992-2 the factor k_1 specified in Sec. 6.8.7 of EN 1992-2 may be taken equal to 1.0.

5.6.4 Simplified fatigue verification for concrete

5.6.4.1 For wind turbine support structures a detailed analysis for concrete under compressive loading is not required, if the maximum stress calculated for the highest fatigue loads in the load spectrum is less than $\sigma_{c,lim}$.

5.6.4.2 $\sigma_{c,lim}$ shall be calculated from the selected method for fatigue verification as prescribed in [5.6.3]. $\sigma_{c,lim}$ shall be taken equal to the maximum allowable stress calculated for the total number of cycles of the load effects which governs the stresses in the particular detail under consideration. For this calculation the minimum stress shall be taken equal to zero.

5.6.4.3 A stress gradient factor can be applied as according to basic design standard if applicable.

5.6.4.4 The damage components of the load effects from erection conditions without the RNA shall also be considered.

5.6.5 Fatigue strength of partially loaded areas

5.6.5.1 Independently of the method applied for concrete fatigue design the increase in fatigue strength due to confinement in partially loaded areas shall be limited as prescribed in DNVGL-ST-C502 [6.20.7] and [6.20.8].

Guidance note:

This rule apply both for designs where DNVGL-ST-C502 has been selected as the basic design standard and for designs where EN 1992-1-1 has been selected as the basic design standard.

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5.7 Accidental limit states

For concrete support structures the earthquake load combinations shall be considered in the accidental limit state. It shall further be noted that for wind turbine support structures the impact from a drifting service vessel is considered an abnormal load as described in DNVGL-ST-0437 [4.2.10] and [4.5.8]. Accidental service vessel impact is therefore handled as ULS.

5.8 Serviceability limit states

5.8.1 Combination of actions

5.8.1.1 For the verifications of the serviceability limit states (SLS) the combinations of actions according to [3.10] are required.

5.8.1.2 Permanent actions shall be included with a load factor of $\gamma_m = 1.0$.

5.8.1.3 Load effects should in SLS be determined based on theory of elasticity. If this is not applied to the validity of the chosen methods shall be documented.

5.8.2 Deformation analysis

If no special requirements arise from operation of the turbine, a limitation of deformations is not necessary. Regarding gapping underneath steel tower flanges see [5.9.2.1].

5.8.3 Natural frequency analysis

5.8.3.1 For towers of reinforced and prestressed concrete, load-dependent stiffness reduction due to cracking shall be considered for the estimation of the natural frequencies of the structure. For this calculation, stabilized crack pattern shall be assumed for the complete tower.

5.8.3.2 The verification of load-dependent stiffness reduction can be omitted for the calculation of the natural frequencies when decompression is verified for the LDD 10^{-2} load, see [3.10].

5.8.3.3 Estimates for Young's modulus of the concrete shall be carefully selected considering both an upper and a lower limit on the natural frequency, and considering for example variations in aggregate stiffness.

5.8.4 Stress limitation

5.8.4.1 For reinforced concrete and prestressed concrete, the concrete compressive stresses for the characteristic extreme load shall be limited to $0.6 f_{ck}$ for design according to DNVGL-ST-C502 and $0.6 f_{ck}$ for design according to EN 1992-1-1.

5.8.4.2 In addition concrete compressive stresses under permanent loads (for example own weight and prestressing, see [3.3]) shall be limited to $0.45 f_{ck}$ for design according to DNVGL-ST-C502 and $0.45 f_{ck}$ for design according to EN 1992-1-1.

5.8.4.3 In order to avoid permanent deformations, the strain in reinforcement shall be calculated for the characteristic extreme load and it shall be substantiated that this strain does not exceed the strain corresponding to $0.9 f_{yk}$.

5.8.4.4 Regarding stress limitations in prestressing tendons the requirements in the basic design standard apply.

5.8.5 Crack control

5.8.5.1 For crack control according to DNVGL-ST-C502 the LDD 10^{-4} load shall be applied together with the crack width requirements listed in the standard and the calculation method specified in DNVGL-ST-C502 App.E.

Let ϵ_{sm} denote the mean principal tensile strain in the reinforcement over the crack's influence length at the outer layer of the reinforcement. Let ϵ_{cm} denote the mean stress-dependent tensile strain in the concrete at the same layer and over the same length as ϵ_{sm} .

For estimation of $(\epsilon_{sm} - \epsilon_{cm})$ the following expression shall be applied:

$$\left(\epsilon_{sm} - \epsilon_{cm} \right) = \frac{\sigma_s}{E_{sk}} \cdot \left(1 - \beta_s \cdot \frac{\sigma_{sr}}{\sigma_s} \right)$$

where:

E_{sk} characteristic value of Young's modulus of steel reinforcement.

σ_s	the stress in reinforcement at the crack calculated for the actual load.
σ_{sr}	the stress in reinforcement at the crack calculated for the load for which the first crack is developed. The tensile strength of the concrete to be used in this calculation is the normalised structural tensile strength, f_{tn} , according to DNVGL-ST-C502 Sec.4 Table C1.
σ_{sr}	$\leq \sigma_s$
β_s	0.4.

Guidance note:

For guidance on how to calculate the free shrinkage strain of the concrete, ϵ_{cs} , see NS 3473:2003 Section A9.3.2.

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5.8.5.2 When the formula in DNVGL-ST-C502 [6.15.3.6] for the nominal crack width is used, the value for c_2 shall be taken as given below:

c_2 = actual nominal concrete cover to the outermost reinforcement (e.g. stirrups).

5.8.5.3 For design according to DNVGL-ST-C502 crack widths shall be calculated in accordance with the method described in DNVGL-ST-C502 [6.15.8] and DNVGL-ST-C502 App.E.

5.8.5.4 For design according to EN 1992-1-1 the crack width calculation method given in EN 1992-1-1 shall be used. The crack width requirements shall in general be taken according to Table 7.1N in EN 1992-1-1 together with the LDD 10^{-2} load for "quasi-permanent load combination" and the LDD 10^{-4} load for "frequent load combination". For members with normal reinforcement in exposure class XD1, XD2, XS1 and XS3, and for members with unbonded tendons in exposure class XC2, XC3, XC4, XD1, XD2, XS1, XS2 and XS3, the crack width requirement shall instead be 0.2 mm.

5.8.5.5 For design according to EN 1992-1-1 k_t shall always be taken as $k_t = 0.4$ due to the highly dynamic loading from a wind turbine.

5.9 Detailing of reinforcement and tower anchorage

5.9.1 Positioning of reinforcement

5.9.1.1 Concrete beams and slabs should always be designed with shear reinforcement sufficient to withstand all fatigue loading.

5.9.1.2 All shear reinforcements and stirrups should be anchored outside the main reinforcement.

5.9.2 Anchor bolt cages and embedded steel cans

5.9.2.1 For cast in steel bolts anchored by a steel anchor flange the bolts shall be post-tensioned and the bolts shall be isolated from the concrete if the reactions are assumed transferred to the concrete at the anchor flange. By isolation is meant that there is no direct contact between the bolts and the concrete and that the bolts thereby can move freely along the entire length of the bolts. It should be secured that there is no decompression underneath the tower bottom flange for the characteristic extreme load according to [3.10] when the tension force is reduced for losses from for example lock-off loss, creep, shrinkage, and relaxation.

5.9.2.2 For embedded steel cans the load transfer from the steel element to the concrete structure shall be carefully considered. Special attention shall be paid to the risk of punching shear from the embedded steel cans and from other embedded steel elements. The risk of punching shear shall be secured by sufficient distance from the surface of the concrete structure and possibly by shear reinforcement. Also the risk of splitting shall be considered around the embedded elements.

5.10 Corrosion control and electrical earthing

5.10.1 Corrosion control

5.10.1.1 Requirements for corrosion protection arrangement and equipments are generally given in [4.16]. Special evaluations relevant for offshore concrete structures are given in [5.10.1.2] and in DNVGL-ST-C502 [6.18].

5.10.1.2 Reinforcement and prestressing tendons are adequately protected by the concrete itself, i.e. provided adequate coverage and adequate type and quality of the constituent materials. However, rebar portions freely exposed to aggressive environment in case of concrete defects and embedment plates, penetration sleeves and various supports (e.g. appurtenances) will require corrosion protection.

Guidance note:

It is recommended always to install cathodic protection for an offshore wind turbine concrete structure. The corrosion protection may be combined with the electrical earthing system for the wind turbine structure, see [5.10.2].

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5.10.2 Electrical earthing

5.10.2.1 All metallic components in a wind turbine concrete structure including appurtenances shall have equipotential bonding and electrical earthing in order to protect against potential differences, stray currents and lightning. Documentation for this shall be included in the design documentation.

Guidance note:

Often the transfer resistance for the reinforcement in an offshore concrete structure will be low and could then be used for earthing. If used for earthing the reinforcement should as a minimum be tied with metallic wire at every second crossing and the vertical and horizontal connection should be supplemented by separate electrical connections clamped to the reinforcement at a suitable distance. For large structures with high reinforcement densities, tying reinforcement with metallic wire at every second crossing may not be viable and the required amount of tie wire will then have to be assessed on a case-by-case basis to ensure sufficient contact area. Care must be taken to ensure that the corrosion protection system and the electrical earthing are not in conflict.

For lightning protection see IEC61400-24.

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5.11 Construction


5.11.1 General

5.11.1.1 For design according to DNVGL-ST-C502 construction shall be performed according to DNVGL-ST-C502 Sec.7, if necessary together with other relevant standards as stated in DNVGL-ST-C502 [7.1.2].

5.11.1.2 If the design verification is performed according to EN1992-1-1, the construction shall follow EN 13670-1.

5.11.2 Inspection classes

5.11.2.1 In general, inspection class IC2, normal inspection (see DNVGL-ST-C502 [7.4.2.1]) applies for wind turbine concrete structures. For critical details it shall be considered to apply inspection class IC3, extended inspection.



5.11.2.2 For construction according to EN 13670-1, execution class 2 applies in general for wind turbine concrete structures. For critical details it shall be considered to apply execution class 3.

SECTION 6 GROUTED CONNECTIONS

6.1 Introduction

6.1.1 General

6.1.1.1 The section provides requirements for grouted connections formed by two structural steel components and the grout-filled space between them. The requirements are applicable to grouted connections in structures for support of offshore wind turbines. Requirements are given for grouted tubular connections and grouted conical connections in monopile structures. Requirements are also given for grouted tubular connections in jacket structures with shear keys. For design of other types of grouted connections, reference is made to DNVGL-ST-C502.

6.1.1.2 Grouted tubular connections are structural connections, which consist of two concentric tubular sections where the annulus between the outer and the inner tubular has been filled with grout. Grouted conical connections are structural connections which consist of two concentric conical sections where the conical shaped space between the two cones has been filled with grout. Typical grouted connections used in offshore wind turbine support structures consist of double-skin grouted tubular joints as used in e.g. monopile and jacket foundation and single-skin tubular joints as used in e.g. hybrid gravity-based foundation. Additionally, grout-filled tubes may be used as reinforcement of typically tubular joints in jackets.

6.1.1.3 The principle for grouted connections in monopile structures is illustrated in [Figure 6-1](#). In monopile structures constructed from steel, grouted connections typically consist of pile-to-sleeve connections, each formed as a connection between the pile and a transition piece that bridges the gap between monopile and tower.

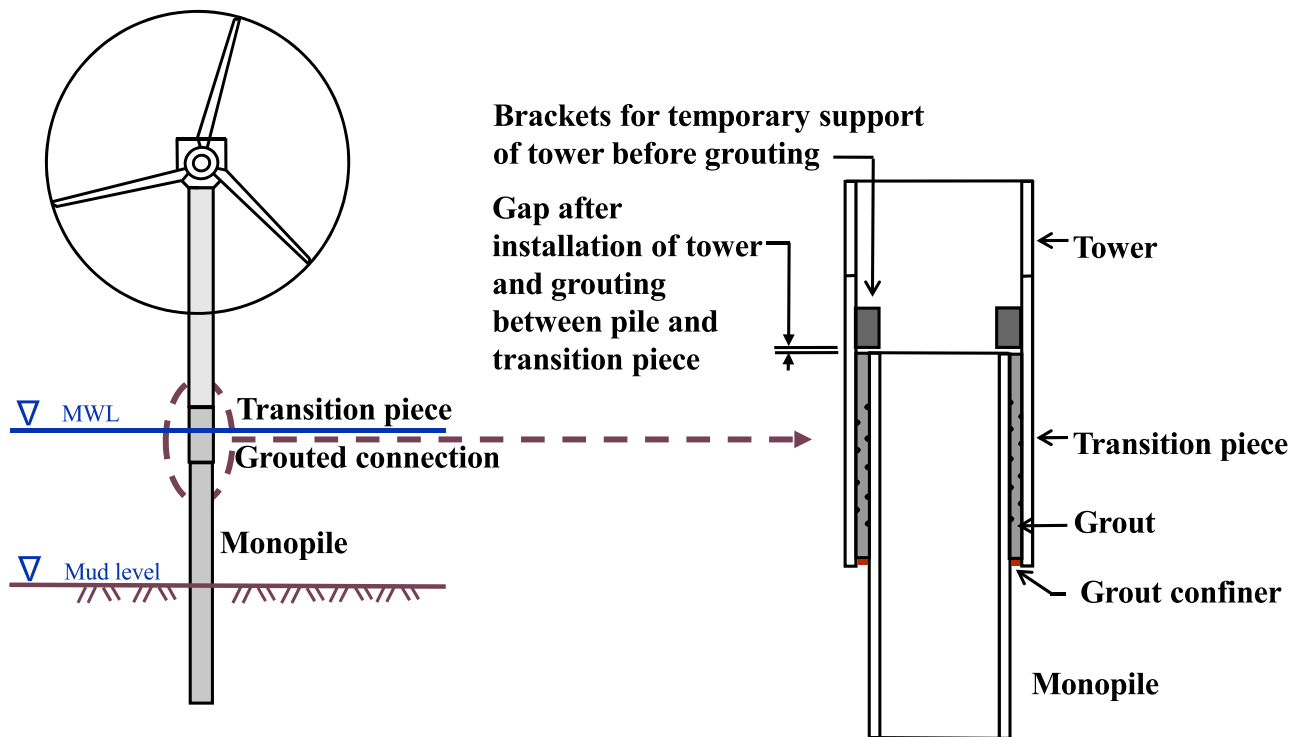


Figure 6-1 Principle of grouted connection in monopile structure

6.1.1.4 Types of grouted connections not covered by this standard shall be specially considered.

6.2 Design principles

6.2.1 General

6.2.1.1 The distance between the mean water level (MWL) and the connection has to be considered in the early design phase since it may have great influence on the behaviour of the connection.

Guidance note:

The location of the connection relative to MWL may influence the shrinkage of the grout, the size of the bending moment in the connection, the fatigue performance of the connection, and the grouting operation.

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6.2.1.2 A grouted connection can be established with or without shear keys. An example of shear keys is given in Figure 6-2.

Guidance note:

In general the application of shear keys improves the ultimate bearing capacity of a grouted connection significantly. Nevertheless shear keys may reduce the fatigue strength of the tubular members and of the grout due to the stress concentrations around the shear keys. Depending on the flexibility of the grouted connection, shear keys used to transfer axial loads will also transfer part of an applied bending moment. This implies that in practice it may be difficult to fully separate axial loads and bending moments when designing the shear keys.

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6.2.1.3 Grouted connections in monopiles may be made with or without shear keys. When the connection is to transfer axial force, the connection shall be conical or it shall be made with shear keys, but it should not combine a conical shape with shear keys. Tubular (cylindrical) grouted connections in monopiles and jacket structures which are to transfer axial force shall always be designed and constructed with shear keys.

Guidance note:

Alternating bending moments in connection with high cyclic loading, implies degradation and loss of bond between grout and steel in tubular grouted connections. This explains why no axial capacity can be counted on in such connections unless they are designed with shear keys.

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6.2.1.4 A grouted connection in a monopile can be constructed with the transition piece placed either outside or inside the foundation pile.

Guidance note:

Traditionally the transition piece is located outside the foundation pile for connections near MWL. This is mainly to be able to mount accessories like boat landings and to paint the structure before load-out. These issues must be paid special attention if the transition piece is placed inside the foundation pile. Locating the transition piece inside the foundation pile rather than outside may not protect the grout from wave action and associated wash-out during the curing of the grout. Full protection from wave action and associated washout during the curing of the grout will require fitting of a protective cover of the exposed grout at the top of the grouted connection.

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6.2.1.5 Grouted connections in jacket structures are either grouted connections between jacket legs and preinstalled piles, or grouted connections between jacket sleeves and post-installed piles. These connections shall always be designed and constructed with shear keys.

Guidance note:

Special design solutions for cylindrical grouted connections without shear keys might be possible for certain boundary conditions. For further information reference is made to DNVGL-RP-0419.

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6.2.1.6 Steel members belonging to the grouted connection, as e.g. shells and shear keys shall be designed according to [Sec.4](#). Reinforcement if present shall be designed according to [Sec.5](#).

Guidance note:

Reinforcement may be used to increase the strength and durability of the grout. The aim should be to provide a shear reinforcement of the grout by the introduction of longitudinal rebars.

This may be advantageous in for example designs where the pile sleeve is attached to the jacket leg by an upper and lower yoke plate. Additional guidance on these types of design is given in NORSOK N-004.

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6.2.1.7 Local buckling in steel tubes shall be considered.

6.2.1.8 Installation tolerances have to be taken into account for design verification and have to be specified in advance.

6.2.1.9 All relevant factors which may influence the bearing capacity of a grouted connection shall be adequately considered and accounted for in the design.

Guidance note:

The strength of grouted connections may depend on factors such as:

- grout strength and Young's modulus
- resistance against wear
- tubulars and grout annulus geometries
- application of mechanical shear keys
- grouted length to pile-diameter ratio
- surface conditions of tubular surfaces in contact with grout
- grout shrinkage or expansion
- dry or wet conditions
- load history (mean stress level, stress ranges, number of cycles)
- environmental conditions/movements during grouting operation
- abrasive wear of contact surfaces.

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6.2.1.10 In general the design procedure can follow an analytical path or a numerical path based on an appropriate standard. Requirements for application of this analytical method are given in [C.1]. In case these requirements lead to a non-favorable design solution a numerical analysis by means of a finite element analysis (FEA) can be used for assessment of the capacity of grouted connections. Guidance for finite element analysis is given in DNVGL-RP-0419 and for verification in [C.2]. Several parameters required for assessment of capacity of grouted connections by use of finite element analysis are encumbered with uncertainty. Uncertainties are associated with element types, element mesh in the region of the highest stresses, friction coefficient, characteristics of the grout material, material modelling, contact formulation, and convergence criterion. Therefore, before using finite element analysis for assessment of capacity of grouted connections, the analysis methodology shall be calibrated to capacities for well-known cases or to reliable test data wherever such data exist.

Guidance note:

It is recommended to limit the nominal contact pressure to 1.2 MPa in design, thereby to limit the consequences of wear. Abrasive wear is caused by a long-term loading of a kind similar to the one that leads to fatigue. Therefore a load factor γ_f equal to 1.0 can be used. For documentation of long-term durability, it is recommended to perform testing of abrasive wear of the grout used for grouted connections of wind turbine structures, when the grout is of a kind for which no inservice experience exists. It is also recommended to perform calculations of the resulting abrasive wear.

The following assumptions are prerequisites for the analytical design procedure:

- All shear keys are assumed to have the same distance s and height h .
- The number of shear keys on the TP and the number of shear keys of the pile differ by one.
- The grout layer should be homogeneously filled with grout material.

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6.2.1.11 Abrasive wear of contact surfaces between steel and grout, subject to relative sliding, is a failure mode that needs to be considered in design. The wear itself is defined as a loss of material in units of weight or volume. The wear may be considered to be proportional to the contact pressure and to the relative sliding length. The sliding length itself is proportional to the alternating bending moment and thus to the contact pressure.

6.2.1.12 The minimum grout thickness is required to be 40 mm considering all manufacturing and installation tolerances.

6.3 Grout materials and material testing

6.3.1 General

6.3.1.1 The designer has to specify the grout properties for final state of the grouted connection. Furthermore the designer has to define the required grout strength for installation of tower and turbine.

6.3.1.2 The grouting contractor has to prove that they can achieve the required fresh grout properties specified by the material manufacturer (e.g. open time, flowability, pumpability, etc.) during grouting operation. Project and site-specific conditions have to be considered.

6.3.1.3 The quality of the in-situ grout material properties is the responsibility of the material manufacturer. The grouting contractors should be evaluated by the material manufacturer to insure that they can perform the grouting operation in accordance with generic method statements and/or guidelines from the material manufacturer in order to achieve the stated material requirements.

6.3.1.4 The pumping, method of application, as well as the quality control of the mixing, curing and placement process offshore can have a significant impact on the final as-built performance of the material. It is therefore of utmost importance that grouting contractors adhere to generic method statements and/or guidelines by the material manufacturer.

6.3.2 Grout material

6.3.2.1 For the grout material used for the grouted connection a grout certification for structural grout according to [5.3.4] of this standard is required. This grout certification contains material testing, relevant test methods and a shop approval.

A certificate for the grout material according DNVGL-ST-C502 [9.5] satisfies both grout certification and the application offshore outlined in [5.3.4] when applied within the application limitations outlined on the type approval certificate.

6.3.2.2 The compressive strength of the grout for grouted connections shall comply with [5.3.4].

Guidance note:

Note that the empirical formulas for characteristic interface shear capacity in [C.1.4] and [C.1.5] are based on characteristic compressive strength on 75 mm cubes, $f_{ck,cube75}$.

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

6.3.2.3 Additionally, the variation of results that has to be expected for in-situ conditions offshore has to be considered in the design for high strength grout by an additional including allowance. The characteristic compression strength, f_{ck} , shall be converted to characteristic in-situ compression strength, f_{cn} , by means of the following formula:

$$f_{cn} = f_{ck} \left(1 - \frac{f_{ck}}{600} \right)$$

where:

f_{ck} = characteristic grout cylinder strength [MPa].

Guidance note:

f_{ck} is the characteristic compressive strength according to EN 1992-1-1. In the context of DNVGL-ST-C502 f_{ck} is equivalent to f_{ck} .

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

6.3.2.4 The characteristic direct tensile strength, f_{tk} , of the grout is defined as the mean value of the direct tensile strength. The characteristic direct tensile strength shall be estimated with at least 75% confidence. In lack of available test data for direct tensile strength, strength values for the characteristic direct tensile strength may be derived from flexural tensile strength, also known as the modulus of rupture, as established from tests on prisms tested in accordance with ASTM C348 or EN 196-1. Further details regarding determination of f_{tk} can also be found in DNVGL-ST-C502.

6.3.2.5 Additionally, the variation of results that has to be expected for in-situ conditions offshore has to be considered in the design for high strength grout by an additional allowance.

The characteristic tensile strength, f_{tk} , shall be converted to characteristic in-situ tensile strength, f_{tn} , for use in the design calculations by means of the following formula:

$$f_{tn} = f_{tk} \left(1 - \left(\frac{f_{tk}}{25} \right)^{0.6} \right)$$

where

f_{tk} = characteristic direct tensile strength of the grout [MPa].

6.3.2.6 The design strengths of the grout material in compression and tension are found by dividing the characteristic in-situ strengths f_{cn} and f_{tn} by a material factor:

$$f_{cd} = f_{cn} / \gamma_m$$

$$f_{td} = f_{tn} / \gamma_m$$

Requirements for the material factor γ_m are specified with each individual application in [6.4.1].

6.3.2.7 The resistance of grout material related to fatigue loads can be affected by the environmental conditions (dry/wet, temperature, early age cycling) and the methodology used for designing. More information will be given in the appropriate subsection of [6.6].

The grout materials used in the numerical approach for grouted connection according to DNVGL-RP-0419 shall be taken according to DNVGL-ST-C502 or Model Code 2010 depending on the standard underlying the verification procedure.

6.3.2.8 In case that further results from accordant studies or grout specific experimental data for the fatigue resistance of grout are available, these results may be taken as a basis for the design of the grouted connection.

6.3.3 Time-dependent grout properties

6.3.3.1 For project specific offshore conditions the time-dependent grout strength for different states of construction while hardening (e.g. early age cycling verification) has to be taken into account.

The development of compressive strength depends on several factors (e.g. environmental temperature, type of cement). Especially for early age movements and low temperatures ($T \leq 20^\circ\text{C}$) hardening tests shall be performed to determine the strength development for the expected offshore conditions. These tests are part of the grout certification for the grout material (see [5.3.4]).

6.3.3.2 Creep and shrinkage: The influence of autogeneous shrinkage has to be minimized by using appropriate grout materials. The influence of shrinkage should be considered in the design and verification.

6.3.4 Early age cycling

6.3.4.1 Only limited relative movement between the grouted steel members is allowed during the initial curing of the grout. The possible movements between the inner and the outer tubular steel members during the 24-hour period after grouting shall be determined for the maximum expected sea states during that time, assuming that the grouted connection does not contribute to the stiffness of the system. For pile-to-sleeve connections, the determination shall be based on an on-bottom analysis of the structure with ungrouted piles. If the expected relative movement at the grout-steel interface exceeds 1 mm during the period until significant grout strength is reached, the movement shall be limited to maximum 1 mm by implementing suitable means.

Guidance note:

Early age cycling is still object of research. If movements exceed the limit given above the long-term grout strength should be reduced to a reasonable value. The definition of an additional safety factor due to movements in curing phase has to be proven by test results.

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

6.3.5 Material testing

6.3.5.1 For some applications, specific properties of the grout may require validation by testing. For example, if hydration during curing of the grout may introduce unacceptable thermal strains in the steel or in the grout, it shall be confirmed that the maximum temperature rise caused by the hardening process is within acceptable limits.

6.3.5.2 All grout properties have to be tested and conformed by an independent test lab taking into account project specific parameters, such as geometry of grouted connection, environmental conditions etc. Specific grout properties (e.g. compressive strength, flowability, air void content) have to be checked on each batch of the grout during grout installation.

6.3.6 Mock-up test

6.3.6.1 The performance of the grout material shall be documented by a full scale mock-up test carried out with relevant equipment to be applied during grouting operations.

Guidance note:

It is recommended to perform the mock-up test at an early stage in the grout certification process to ensure that the grout material may be applied under the project specific boundary condition.

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

6.3.6.2 Procedure for large scale mock-up test: The mock-up test shall directly correspond to a grouting procedure for a specific application. The test-setup shall reflect the actual conditions and equipment to be used at the site including the grout mixer and pump, pumping height and hose with a representative nominal bore diameter and length to assess pumpability of the material. The mock-up test shall demonstrate that the material maintains pumpability over the likely duration of the operation including possible pauses due to blockages or equipment failures. The most challenging placement configuration expected offshore shall be reflected in the test plan including contingency procedures. Appropriate material testing shall be conducted during the test and complete filling of the intended volume shall be demonstrated after hardening. The precise requirements with regard to the mock-up test depends on the grouting operation (and procedure) under consideration. While performing the mock-up test a continuous quality control of the grout material according to [6.3.7] shall be done.

6.3.6.3 The requirement may be waived if geometry, equipment, and personnel are identical or very similar to previous projects executed by the same contractor for which a mock-up test has already been successfully conducted.

6.3.7 Quality control of grout material used for grouted connection

6.3.7.1 Samples for quality control during offshore grouting of grout mixed in a continuous process (i.e., not batch mixing) shall preferably be taken from the emerging return grout. If this is not possible, other means of monitoring the quality of the return grout shall be provided. Samples of pre-packed blended grout for quality control during offshore grouting should represent the grout conditions on site.

6.3.7.2 Testing of specimens from grout sampling offshore shall be carried out in order to verify the characteristic compressive strength of the grout. The characteristic compressive strength is normally defined as the compressive strength after 28 days curing at 20°C submerged in water. Additional testing under the offshore environmental conditions may be required, if the grout shall be subjected to loading before the characteristic design strength has been achieved, for example due to installation of other structures or due to wave and wind loading before 28 days have passed, the assumed allowable grout strength at the time of the loading shall be documented.

6.3.7.3 The required test methods and testing frequency for offshore quality control testing is stated in DNVGL-ST-C502 [7.6].

Guidance note:

The specified minimum requirement for the number of test specimens for compressive strength usually implies that five test specimens are obtained from the annulus of each grouted structure. When this is the case, it is acceptable to estimate the grout strength as the average strength over all obtained samples, i.e. over a number of specimens equal to five times the number of grouted structures, provided that it is demonstrated statistically that the compressive strengths obtained from the tests on these samples belong to the same population.

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

6.3.7.4 An external quality control should be performed by an accredited laboratory or by a laboratory follow comparable procedures.

6.4 Structural design

6.4.1 Design material strength

6.4.1.1 In design by calculation according to this standard, the design material strength of the grout material shall be determined by applying a material factor γ_m .

6.4.1.2 For design according to the analytical design approach as given in [C.1] the material factors, γ_m , for grout material in limit states ULS, FLS, ALS, SLS shall be taken according to Table 6-1.

Table 6-1 Requirements for material factor γ_m for grout depending on type and load case

<i>Type of grouted connection</i>	<i>Load case</i>	<i>Material factor γ_m</i>
Cylindrical grouted connections with shear keys	ULS	2.0
Conical grouted connections	ULS	1.5
Cylindrical grouted connections with shear keys	FLS	1.5

Type of grouted connection	Load case	Material factor γ_m
Cylindrical grouted connections with shear keys	ALS	1.7
Conical grouted connections	ALS	See Table 5-2 in [5.4.1]
All	SLS	1.0

6.4.1.3 For design according to the numerical design approach as given in [C.2] based on DNVGL-ST-C502 the material factors, γ_m , for grout material and limit states ULS, FLS, ALS, SLS are given in Table 5-2 in [5.4.1].

6.4.1.4 For design according to the numerical design approach as given in [C.2] based on EN 1992-1-1 the material factors, γ_m , shall be taken according to [5.4.1].

In design by calculation according to EN 1992-1-1 the material factor γ_m for material strength of grout material is to be applied as a divisor on the product of f_{ck} and α_{cc} as specified in EN 1992-1-1 Sec. 3.1.6.

6.4.2 Load transfer through grouted connection

6.4.2.1 Grouted connections in wind turbine support structures shall be designed for the ULS and the FLS loads and load combinations as specified in Sec.3.

6.4.2.2 Jackets with preinstalled piles often have large tolerances. When such large tolerances are in place, the moment due to eccentricity associated with the tolerances should be included in design in addition to the design bending moment.

6.4.3 Effect of installation/manufacturing tolerances

6.4.3.1 The effect of installation tolerances on load distribution shall be investigated and taken into account for verification.

6.4.3.2 The effect of manufacturing tolerances on load distribution shall be investigated and taken into account for verification.

Guidance note:

Special attention has to be given to the height of shear key h , since the characteristic interface capacity is directly dependent on this value. For verification of limit states the minimum value should be taken. The value is calculated by the nominal height minus the manufacturing tolerance.

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

6.4.3.3 For grouted connections designed with shear keys, a fabrication mispositioning of the shear keys equal to +/- 10 mm relative to nominal position is within the mispositioning accounted for in the design procedures in [6.5] and [6.6].

6.5 Ultimate limit states

6.5.1 General

6.5.1.1 If DNVGL-ST-C502 has been chosen as the basic design standard the ULS verification shall follow this standard.

6.5.1.2 In case of using EN 1992-1-1 as the basic design standard, the design shall be performed based on this standard

6.5.1.3 In ULS the load effects can be determined according to either linear elastic analysis or plastic analysis as described in the basic design standard.

6.5.1.4 The increase in the internal forces and moments through non-linear influences (e.g. second-order theory, crack formation) shall be taken into account. It may be determined from a quasi-static calculation. This is implicitly accounted for in the analytical design approach.

6.5.1.5 For design and verification of steel members belonging to grouted connections (e.g. tubes, shear keys), reference is made to [Sec.4](#).

6.5.1.6 The analytical expressions for contact pressure in [\[C.1\]](#) can be used as basis for calibration of the finite element analysis methodology in absence of other reliable data. The calibration analysis shall be performed for a geometry for which the analytical expression for contact pressure is considered to be valid. Then the derived analysis methodology can be used for assessment of the capacity of other grouted connections. A calibrated analysis methodology can also be used for assessment of local stresses and local capacities.

6.5.2 Tubular and conical grouted connections in monopiles without shear keys

6.5.2.1 Requirements for tubular and conical grouted connections in monopiles without shear keys are given in [\[C.1.1\]](#).

6.5.2.2 For tubular grouted connections it is a prerequisite that the connection shall not transfer axial force.

6.5.2.3 For conical grouted connections it is a prerequisite that the cone angle is less than 4° . Additional special provisions for conical grouted connections in monopiles without shear keys are given in [\[C.1.2\]](#).

Guidance note:

Elastomeric bearings can be used as an alternative for transfer of axial force; however, elastomeric bearings are not likely to be as attractive as conical connections in this respect.

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

6.5.3 Tubular grouted connections in monopiles with shear keys

6.5.3.1 The purpose of the grouted connection is to transfer pressures between the two tubes and thereby provide capacity against bending moments. The bending moment capacity consists of a component formed by radial contact pressures, a component formed by horizontal shear resistance between steel and grout, a component formed by vertical shear resistance between steel and grout, and a component formed by moment resistance of shear keys. The purpose of the grouted connection is also to transfer the vertical loads from the transition piece to the monopile. The vertical load capacity is achieved by the shear keys.

6.5.3.2 The shear keys shall be placed in the central region of the grouted connection as indicated in [Figure 6-2](#), i.e. the region where the grouted connection can be reckoned not to open up significantly during bending moment loading. Possible vertical shear keys to provide capacity against torsional loading should also be placed in this central region, but may be placed outside the region if necessary, for example when fabrication constraints make it difficult to place them inside the region. The central region extends over half the effective grout length L_g , is centered in the grouted connection midpoint and has a distance $L_g/4$ to either end of the effective part of the grout.

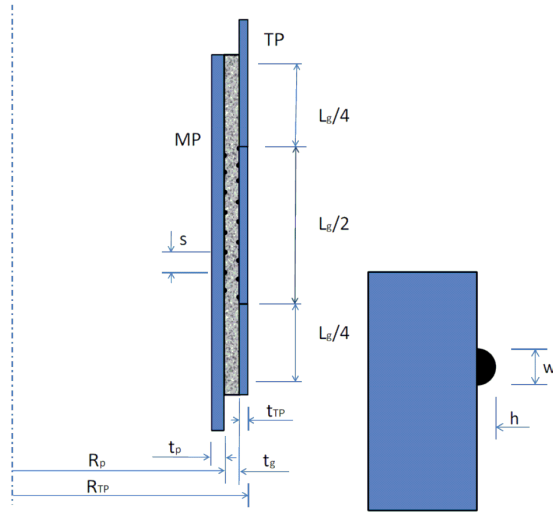


Figure 6-2 Definition of symbols for grouted connection with shear keys in monopile

6.5.3.3 Geometrical requirements for application of the analytical approach are given in [C.1.3.9] to [C.1.3.12].

6.5.3.4 The design criterion for the grouted connection with shear keys is:

$$F_{V1Shk,d} \leq F_{V1Shk cap,d}$$

where F_{V1Shk} is the action force per unit length along the circumference, owing to bending moment and vertical force and transferred to the shear key and $F_{V1Shk cap,d}$ is the design capacity per unit length of one shear key. Further information can be found in [C.1.3].

6.5.3.5 Additionally the following requirement for the maximum nominal radial contact pressure shall be fulfilled:

$$p_{nom,d} \leq 1.5 \text{ MPa}$$

This requirement for the nominal radial contact pressure can be waived if a detailed FE analysis is performed in accordance with [C.2] and DNVGL-RP-0419.

6.5.3.6 For grouted connections subjected to torque, vertical shear keys shall be used to provide adequate torsional capacity. The shear keys shall be placed as specified in [6.5.3.2]. The torsional resistance resulting from permanent weight of the structure above the transition piece, and including the transition piece, can be counted on.

Guidance note:

Normally it is expected that rather few vertical shear keys are needed in grouted connections in monopile structures to achieve sufficient torsional capacity. Examples of shear key layouts are given in Figure 6-3.

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

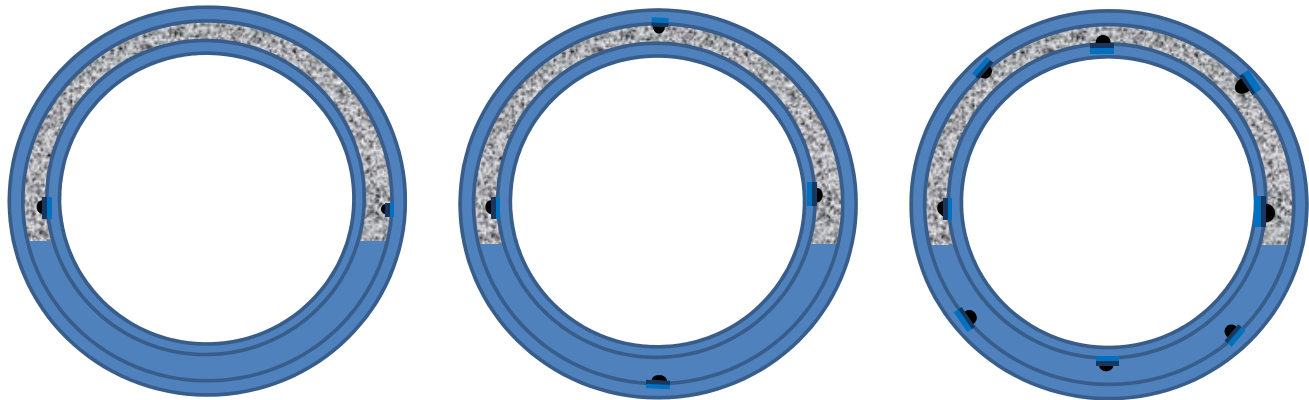


Figure 6-3 Examples of vertical shear key layouts for torsional capacity

6.5.3.7 The design criterion for the vertical shear keys is:

$$F_{H1Shk,d} \leq F_{H1Shk\,cap,d}$$

where F_{H1Shk} is the action force per unit length, owing to torque moment and transferred to the shear key and $F_{H1Shk\,cap,d}$ is the design capacity per unit length of one shear key. More details on the verification procedure for tubular grouted connections in monopiles with shear keys are given in [C.1.5].

6.5.4 Tubular grouted connections in jacket structures with shear keys

6.5.4.1 For design of tubular grouted connections in jacket structures, it is important to minimize cracking of the grout caused by cyclic axial loading with load reversal. Without load reversal or with axial load mainly in only one of the two axial directions, the cracking in the grout will be such that the connection will still be capable of transferring load. With axial load in one direction, some redistribution of loads on shear keys can be assumed to take place and the load effect can be derived from an assumption of uniform load on the shear keys. When the assumption of axial loading in only one direction is not fulfilled and there is a smaller axial load in the opposite direction due to load reversal, caution shall be exercised to ensure that cracking does not take place for this smaller axial force in the opposite direction.

Guidance note:

The requirement for caution to ensure that cracking does not take place for the smaller axial force in the opposite direction is assumed to be fulfilled when the design procedure specified in [6.6.1] and [6.6.3] is followed.

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

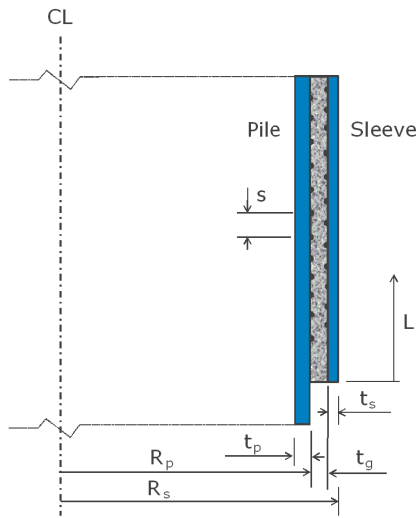


Figure 6-4 Grouted pile-to-sleeve connection in jacket structure

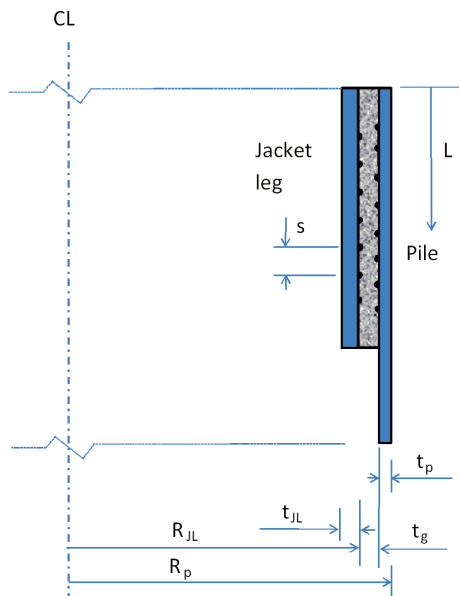


Figure 6-5 Grouted leg-in-pile connection in jacket structure

6.5.4.2 For connections involving post-installed piles, the region significantly affected by the bending moment is the region from a level half an elastic length (see [C.1.4.12]) above the base of the connection and downwards. To avoid that shear keys will initiate cracks in the grout in this region due to dynamic bending moment, it is recommended that no shear keys be placed in this region.

6.5.4.3 For connections involving pre-installed piles, the region significantly affected by the bending moment is the region from a level half an elastic length below the top of the connection and upwards. To avoid that

shear keys will initiate cracks in the grout in this region due to dynamic bending moment, it is recommended that no shear keys be placed in this region.

6.5.4.4 The design criterion for the grouted connection with shear keys is:

$$F_{V1Shk,d} \leq F_{V1Shk cap,d}$$

where F_{V1Shk} is the action force per unit length along the circumference, owing to bending moment and vertical force and transferred to the shear key. Further information may be found in [C.1.3].

6.5.4.5 The following requirement for the nominal contact pressure due to bending and transverse shear force shall be fulfilled:

$$p_{nom,d} \leq 1.5 MPa$$

6.5.4.6 This requirement for the nominal radial contact pressure can be waived if a detailed FE analysis is performed in accordance with [C.2] and DNVGL-RP-0419.

Guidance note:

Alternatively the grout may be reinforced to add additional strength and durability in particular against shear failure.

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

6.6 Fatigue limit states

6.6.1 General

6.6.1.1 All load and stress fluctuations imposed during the design life of the grouted connection and which are significant with respect to fatigue evaluation shall be taken into account when the long term distribution of stress ranges is determined. Reference is made to DNVGL-ST-0437.

Guidance note:

Statistical considerations for loads of a random nature are required for determination of the long term distribution of fatigue loading effects. Deterministic analysis or spectral analysis may be used. The method of analysis should be documented.

The effects of significant dynamic response should be properly accounted for when stress ranges are determined.

Special care shall be taken to adequately determine the stress ranges in structures or members excited in the resonance range.

The amount of damping assumed shall be appropriate to the design.

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

6.6.1.2 Fatigue design may alternatively be undertaken by utilizing methods based on fatigue tests and cumulative damage analysis. Such methods shall be appropriate and shall be adequately documented.

6.6.1.3 For structures subjected to multiple stress cycles, it shall be demonstrated that the structure will endure the expected stresses during the required design life.

6.6.1.4 Installation tolerances shall be considered for design verification.

6.6.1.5 The verification shall be performed by means of detailed cycle count matrix in form of a Markov matrix including range, mean and cycle number values.

6.6.1.6 The cumulative damage can be calculated according to Palmgren-Miner's rule

6.6.1.7 The characteristic number of cycles to failure is given in terms of a characteristic S-N curve. The characteristic S-N curve of a structural detail shall be applicable for the material, the structural detail, the state of stress considered and the surrounding environment. S-N curves are to take into account material thickness effects as relevant.

6.6.1.8 For a given stress level, the characteristic number of cycles to failure is defined as the 5% quantile in the distribution of the number of cycles to failure, i.e. the characteristic number of cycles to failure is the number of cycles that provide a 95% survival probability.

6.6.2 Conical grouted connections in monopiles without shear keys

6.6.2.1 Calculation of fatigue life at varying stress amplitude can be based on cumulative linear damage theory. The stresses due to cyclic actions may be arranged in stress blocks (action effect blocks) each with constant amplitude stress and a corresponding number of stress cycles, n_i . A minimum of 8 blocks is recommended. The design criterion is:

$$DFF \cdot \sum_{i=1}^k \frac{n_i}{N_i} \leq 1.0$$

where:

k = total number of stress blocks

n_i = number of stress cycles in stress block i

N_i = characteristic number of cycles to failure at the constant stress amplitude of stress block i

DFF = design fatigue factor.

6.6.2.2 The design fatigue factor DFF shall be taken as 3.0 in case the design procedure is following DNVGL-ST-C502.

6.6.2.3 In case of using EN 1992-1-1, the fatigue design shall be performed based on Model Code 2010 with the amendments prescribed in [C.2.4.5] and a design fatigue factor DFF shall be taken as 1.0.

6.6.2.4 The characteristic number of cycles to failure can be estimated from test data obtained from fatigue tests on appropriate grout specimens. In absence of representative test data, the characteristic number of cycles to failure can be calculated according to the prescriptive method given in [C.2.7.1].

6.6.2.5 When the characteristic S-N curve shall be estimated from test data, it shall be estimated with at least 75% confidence.

6.6.2.6 Design of grouted connections against fatigue failure according to the procedure and the requirements given in [6.6.2] will meet the requirements for fatigue design set forth in IEC 61400-1.

6.6.2.7 More details on the verification procedure for conical grouted connections in monopiles without shear keys are given in [C.2.7].

6.6.3 Tubular grouted connections with shear keys, general

6.6.3.1 Vertical downward dynamic loading on a shear key will potentially lead to cracking along compression struts in a zone between the shear keys on the pile and on the sleeve (or transition piece). Vertical upward dynamic loading on a shear key will also potentially lead to cracking along compression struts in a zone between the shear keys on the pile and on the sleeve (or transition piece). In order to properly

reflect that load reversal is what is critical with respect to fatigue, load amplitudes are defined as maximum upward and downward deviations from zero load level, i.e. the limit between upward and downward loading, rather than as maximum deviations from the usually non-zero mean load level. There is one maximum upward deviation and one maximum downward deviation per load cycle. To design the grouted connection against fatigue failure in both zones, two long term distributions of load amplitudes defined this way shall be established:

- Long term distribution of amplitude of vertical downward load on shear key. The distribution consists of one downward amplitude value per load cycle, see [Figure C-1](#) and [Figure C-2](#) in [App.C](#).
- Long term distribution of amplitude of upward load on shear key. The distribution consists of one downward amplitude value per load cycle, see [Figure C-1](#) and [Figure C-2](#) in [App.C](#).

Load amplitudes for the total load on the shear keys are defined as maximum deviations from zero load level. When subtracting the effect of permanent load from these amplitudes, a new set of load amplitudes results which are load amplitudes for the environmental load and which are then defined as maximum deviations from zero environmental load. This is illustrated in [Figure C-1](#) and [Figure C-2](#) in [App.C](#) where the solid abscissa is located at zero load level for the environmental loading and the dashed abscissa, which is obtained by a shift to reflect the effect of permanent load caused by deadweight, is located at zero load level for total load.

6.6.3.2 The design criterion is:

$$D = DFF \cdot \sum_{j=1}^k \sum_{i=1}^{n_0} \frac{n_i}{N_i} \leq 1.0$$

where:

- n_0 = total number of stress blocks of constant-amplitude stress
- n_i = number of stress cycles in the i^{th} stress block
- k = number of lateral environmental load directions
- N_i = characteristic number of cycles to failure at the constant stress amplitude of stress block i

The design criterion shall be fulfilled for the following two cases:

- The cumulative damage D is calculated based on the long term distribution of the amplitude of the vertical downward dynamic load on the shear key.
- The cumulative damage D is calculated based on the long term distribution of the amplitude of the vertical upward dynamic load on the shear key.

6.6.3.3 The design fatigue factor DFF shall be taken as 1.0 in case design procedure is following the procedures given in [\[C.1\]](#).

6.6.3.4 In case a numerical verification procedure according to [\[C.2\]](#) is applied the following design fatigue factor DFF shall be taken:

- $DFF = 3$ in case the design procedure is following DNVGL-ST-C502
- $DFF = 1$ In case of using EN 1992-1-1 and Model Code 2010

6.6.3.5 The effect of installation tolerances on the fatigue capacity should be assessed. Reference is made to [\[C.1.6.2\]](#).

6.6.4 Tubular grouted connections in monopiles with shear keys

6.6.4.1 More details on the verification procedure for tubular grouted connections in monopiles with shear keys are given in [C.1.6] and [C.1.7].

6.6.5 Tubular grouted connections with shear keys in jacket structures

6.6.5.1 More details on the verification procedure for tubular grouted connections in jacket structures with shear keys are given in [C.1.8]. Provisions are given for connections in jacket structures with preinstalled piles as well as for connections in jacket structures with piles which are post-installed within pile sleeves or through jacket legs.

6.6.6 Shear keys

6.6.6.1 For fatigue assessment of shear keys with nominal stress concept, the S-N curve designated is:

- Curve E applies, if the design verification is performed according to DNVGL-RP-C203 with nominal stress concept.
- DC 71 applies, if the design verification is performed according to EN 1993-1-9.

6.6.6.2 Curve D or DC 90 applies, if the design verification is performed with structural hot spot concept according to DNVGL-RP-C203 or IIW-1823-07 (Fatigue Recommendation 2007).

6.7 Grouting operations

6.7.1 General

6.7.1.1 The grouting operations of connections are to comply with relevant requirements given in DNVGL-ST-C502 Sec.4, Sec.9 and App.H.

6.7.1.2 The grouting system shall have sufficient venting capacity to enable air, water and surplus grout to be evacuated from the annuli and compartments to be grouted.

6.7.1.3 Injection of grout shall be carried out from the bottom of the annulus. Complete filling of the annulus shall be confirmed by grout overflow of good quality at the top of the grouted connection or at the top outlet hole.

6.7.1.4 Sufficient strength of formwork or similar (e.g. an inflatable rubber seal) shall be ensured.

6.7.1.5 To avoid casting joints in the grout member, the grouting should be carried out in one process. Contingency procedures shall be established prior to grouting and shall be used in the event of blocked primary grout lines, seal failures etc.

6.7.1.6 Sufficient material of acceptable quality shall be available at the start of a grouting operation to enable full filling of grout for the biggest compartment to be grouted.

6.7.1.7 Adequate backup equipment for the grouting process shall be available before the grouting operation is initiated.

6.7.1.8 The temperature of all surroundings (air, water, steel structures etc.) shall be between 5°C and 30°C during the grouting operation unless the structural grout has been validated at temperatures outside this temperature range.

6.7.1.9 In general, piling operations are not to be performed after commencement of pile-grouting operations.

6.7.1.10 All steel surfaces to be in contact with grout shall be clean before grouting. Before positioning of the tubes, the surfaces shall be checked for grease, oil, paint, marine growth etc. and cleaned if necessary.

6.7.2 Inspection of grouting operation

6.7.2.1 During the grouting operation, survey and inspection shall be performed to ensure compliance with the grouting procedure for the specific application.

6.7.2.2 Special care has to be taken if the grouting volume has a boundary to subsea soil and no grout seal is used (i.e. for pre-piled grouted connections). In these cases the volume of grout has to be calculated and has to be measured while grouting. The consistency and sealing of subsea soil, free-falling of grout may influence the grout quality.

6.7.3 Monitoring

6.7.3.1 Parameters considered as important for controlling the grouting operation shall be monitored prior to and during the grouting operation. Records shall be kept of all monitored parameters. These parameters normally include:

- results from qualification tests for grout mix
- results from grout tests during operation
- records of grout density and flow ability at mixer and of total volumes pumped for each compartment or annulus
- records from differential pressure measurements, if applicable
- observation records from evacuation points
- records of grout density at evacuation points or density of return grout, if applicable
- results from compressive strength testing
- temperature of mixed grout
- ambient temperature
- water to powder ratio (batch mixing)
- density readings (continuous mixing).

6.7.3.2 At the end of the grouting operation it shall be ensured that sound, homogeneous grout material is leaving the overflow point and the annulus has been grouted up to the design height.

SECTION 7 GEOTECHNICAL DESIGN

7.1 Introduction

7.1.1 Scope

7.1.1.1 The requirements in this section apply to gravity-based foundations, pile foundations, suction buckets and stability of soil surface.

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

7.2 Geotechnical foundation concepts

7.2.1 General

In this subsection typical geotechnical elements of the foundation structures are described. For steel and concrete elements of the support structures, see [4.1] and [5.2], respectively. See also Figure 1-1. The listing shall be seen as examples and it is not considered a comprehensive list of all possible designs.

7.2.2 Onshore gravity-based foundations

The gravity-based foundations consist of a concrete base slab founded on firm soil or on rock. The objective of the design is to avoid tensile loads (lifting) between the bottom of the foundation and the soil surface by means of sufficient dead loads. The base dimensions can be adjusted depending on the actual soil conditions to achieve sufficient bearing capacity.

7.2.3 Onshore pile foundations

Pile foundations are used in case the upper soil layers do not have a sufficient bearing capacity, or if it is essential to reduce settlements. The loads are transmitted by a combination of skin friction and tip resistance of the piles. These kinds of foundations normally consist of multiple cast-in-place or pre-cast concrete piles connected to a concrete slab (often called piled raft foundations).

7.2.4 Onshore rock anchor foundations

Rock anchor foundations consist of long prestressed steel anchors, deeply fixed in stable rock and directly attached to the lower flange of a tower. The anchors, the rock and the lower tower flange are braced together, thus forming a compact foundation element. The rock prestressed by the anchors, firmly surrounded by similar rock, serves as the element to resist all actions created by the forces on the wind turbine. Regarding rock anchor design reference is made to EN 1997-1 Section 8.

7.2.5 Offshore gravity-based foundations

In addition to [7.2.2], the concrete base slab can be constructed with small skirts, and will for all locations require some form of scour protection. In the offshore industry they are competitive when the environmental loads are relatively modest. To increase the dead loads, ballast can be added. Temporary structures, such as mudmats, also belong to these types of foundations.

7.2.6 Offshore pile foundations

Offshore pile foundations are long, slender flexible steel piles that support a four-legged jacket structure, a three-legged tripod or a hybrid structure. For these types of foundations, the piles are predominantly axially loaded. The pile penetration depth can be adjusted to suit the actual soil conditions.

7.2.7 Onshore and offshore monopile foundations

Offshore monopile foundations are robust, rigid piles with a large diameter. It is a simple design by which the tower is supported directly or through a transition piece. For these types of foundations the piles are predominantly laterally loaded. The limiting condition of this type of foundation is most often the overall deflection and vibration. The pile penetration depth can be adjusted to suit the actual environmental and soil conditions.

7.2.8 Offshore suction bucket foundations

The bucket is installed by means of suction and will behave as a gravity-based foundation after installation, relying on the weight of the soil encompassed by the bucket with a skirt length which may range from a fraction of to approximately the same dimension as the width of the bucket. The stability is ensured because there is not enough time for the bucket to be pulled from the bottom during a wave period. When the bucket is pulled from the soil during the passing of a storm, a cavity will tend to develop inside the bucket between the soil surface and the bucket top. However, the development of such a cavity depends on water to flow in and fill up the cavity and thereby allow the bucket to be pulled up, but the typical wave periods are too short to allow this to happen.

7.3 Soil investigations and geotechnical data

7.3.1 General

7.3.1.1 The soil investigations shall provide all necessary soil data for a detailed geotechnical design. The soil investigations may be divided into geological studies, geophysical surveys and geotechnical soil investigations. Further details can be found in DNVGL-RP-C212, ISO 19901-8 or EN 1997-1 and EN 1997-2.

Guidance note:

A geological study, based on the geological history, may form a basis for selection of methods and extent of the geotechnical soil investigations. A geophysical survey, based on shallow seismic reconnaissance, may be combined with the results from a geotechnical soil investigation to establish information about soil and rock stratification and surface topography or groundwater conditions for an extended area such as the area covered by a wind farm. A geotechnical soil investigation consists of in-situ testing and sampling for laboratory testing of soil and rock.

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7.3.1.2 The extent of ground investigations and the choice of soil investigation methods shall take into account the phase of the project; the type, size and importance of the wind turbine structure; the actual type of soil deposits and the complexity of soil and terrain conditions. The area to be covered by soil investigations shall account for positioning and installation tolerances.

Guidance note:

The line spacing of the seismic survey at the selected location should be sufficiently small to detect all soil strata of significance for the design and installation of the wind turbine structures. Special concern should be given to the possibility of buried erosion channels with soft infill material.

The soil investigation should also be tailored to the design methods intended to be used.

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7.3.1.3 For multiple foundations such as in a wind farm, the soil stratigraphy and range of soil strength properties shall be assessed per foundation location.

Guidance note:

When very homogeneous soil conditions prevail, such assessment may cover a group of foundations within the entire area of a wind farm or it may consist of all the foundations within a sub-area of a wind farm. Such sub-areas are typically defined when groups of wind turbines within a wind farm are widely separated.

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7.3.1.4 Soil investigations should be carried out before the design. However, in the scenario when no soil investigations are available yet when the foundation is designed, conservative assumptions shall be made for the soil properties. These shall be confirmed by soil investigations before the start of construction.

7.3.1.5 Soil investigations shall provide relevant information about the ground to a depth below which possible existence of weak formations will not influence the safety or performance of the wind turbine support structure.

Guidance note:

For design of gravity-based foundations, the soil investigations should extend at least to the depth of any critical shear surface. Further, all soil layers influenced by the wind turbine structure from a settlement point of view should be thoroughly investigated. More thorough testing of the shallow top layers (for instance, by means of CPTs or vibrocores), may also be of relevance. For design of pile foundations, a combination of in-situ testing and borings with sampling should be carried out to sufficient depth. If potential end bearing layers or other dense layers, which may create driving problems are found, this scope should be increased. If no special reasoning is given by the designer, this should be at least half a pile diameter below the pile tip for pile foundations against lateral loads, and at least 3 pile diameters below pile tip for axial loads. Seabed CPTs are also recommended at each leg of an offshore support structure in order to facilitate e.g. the mud mat design. In seismically active areas, it may be necessary to obtain information about the shear modulus of the soil to depths which may have influence on the design in view of shear wave propagation due to earthquakes. If the soil investigations reveal that the relevant ground layers have insufficient strength or stiffness properties, different soil improvement techniques may be considered.

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7.3.1.6 Soil investigations should normally comprise the following types of investigation:

- site geological survey
- topography survey of the soil surface, if applicable
- in-situ testing, for example by cone penetration tests (CPT) or Down-the-hole SPT, pressiometer tests and dilatometer tests
- soil and rock sampling with subsequent static laboratory testing.

They may also comprise, if useful in special circumstances or required by local authorities

- geophysical investigations for correlation with borings and in-situ testing
- shear wave velocity measurements for assessment of maximum shear modulus
- cyclic laboratory testing.

Guidance note:

The extent and contents of a soil investigation program are no straight-forward issue and will depend on the foundation type. The guidance given in these guidance notes therefore forms recommendations of a general nature which the designer may elaborate further on.

An experienced geotechnical engineer who is familiar with the considered foundation concepts and who represents the owner or developer should be present during the soil investigations on the site. Depending on the findings during the soil investigations, actions may then be taken, as relevant, to change the soil investigation program during its execution. This may include suggestions for increased depths of planned borings, suggestions for additional borings, and suggestions for changed positions of borings.

When non-homogeneous soil deposits are encountered or when difficult or weak grounds are identified locally, it may be necessary to carry out more borings and CPTs than the tentative minimum recommended in this subsection.

For solitary wind turbine structures, one boring to sufficient depth for recovery of soil or rock samples for laboratory testing is recommended as a minimum.

In cases where soil conditions are highly varying within small spatial areas and foundations with a diameter of more than 20 m are used, more than one boring or more than one CPT per wind turbine position may be needed.

For cable routes, the soil investigations should be sufficiently detailed to identify the soils of the surface deposits to the planned depth of the cables along the routes. For onshore applications, EN 1997-1 and EN 1997-2 is referred to, if local standards do not prevail.

For offshore wind turbines seabed samples should be taken for evaluation of scour potential.

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7.3.1.7 If the area, where the wind turbine structure shall be installed, is determined to be seismically active and the substructure and foundation will be affected by an earthquake, an evaluation shall be made of the regional and local geology in order to determine the location and alignment of faults, epicentral and focal distances, the source mechanism for energy release and the source-to-site attenuation characteristics. Local soil conditions shall be taken into account to the extent that they may affect the soil motion. The seismic design, including the development of the seismic design criteria for the site, shall be in accordance with recognised industry practice. For details of seismic design criteria, reference is made to ISO 19901-2 and to EN 1998-5

7.3.1.8 For further guidance and industry practice regarding requirements to scope, execution and reporting of onshore soil investigations, and to equipment, reference is made to EN 1997-1 and EN 1997-2 and attendant standards. For offshore soil investigations, and to equipment, reference is made to DNVGL-RP-C212 and ISO 19901-8. Other national and international standards may be considered from case to case, if relevant.

7.3.1.9 The geotechnical investigation at the actual site comprising a combination of sampling with subsequent laboratory testing and in-situ testing shall provide the following types of geotechnical data for all important layers:

- data for soil and rock classification and description
- shear strength and deformation properties, as required for the type of analysis to be carried out
- in-situ stress conditions.

7.3.1.10 The soil and rock parameters provided shall cover the scope required for a detailed and complete foundation design, including the lateral extent of significant soil layers, and the lateral variation of soil properties in these layers.

7.3.1.11 It is of critical importance that soil and rock samples obtained as part of a soil investigation program are of a sufficiently good quality to allow for accurate interpretation of soil and rock parameters for use in design. For requirements to soil and rock sampling quality, soil or rock identification, groundwater measurements and regarding handling, transport and storing of samples, reference is made to ISO 22475-1.

7.3.1.12 The laboratory test program for determination of soil strength and deformation properties shall cover a set of different types of tests and a number of tests of each type, which will suffice to carry out a detailed foundation design.

Guidance note:

For mineral soils, such as sand and clay, direct shear and triaxial tests are relevant types of tests for determination of strength properties. For rocks, unconfined/uniaxial compressive strength tests may be used.

For fibrous peats, neither direct shear tests nor triaxial tests are recommended for determination of strength properties. Shear strength properties of low-humified peat may be determined by ring shear tests.

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7.4 General design requirements

7.4.1 Introduction

7.4.1.1 Design of foundations shall be based on site-specific information, see [Sec.3](#), [\[7.3\]](#) and DNVGL-ST-0437.

7.4.1.2 The geotechnical design of foundations shall consider both the strength and the deformations of the foundation structure and of the soil.

This subsection states requirements for:

- soils around the foundation structure having influence on it once loaded
- soil reactions upon the foundation structure
- soil-structure interaction.

Requirements for the foundation structure itself are given in [Sec.4](#) to [Sec.6](#) as relevant for a foundation structure constructed from steel and/or concrete.

7.4.1.3 A foundation failure is defined as the design point in which the foundation reaches any of its limit states. Examples of such failures are:

- bearing failure
- sliding
- overturning
- pile pull-out
- axial pile capacity
- lateral pile capacity
- large settlements or displacements.

7.4.1.4 The definition of limit state categories as given in [\[2.4\]](#) is valid for foundation design with the exception that cyclic loading may give a reduction to the ultimate bearing capacity in the ultimate limit state (ULS). The partial load and material factors as defined for these limit state categories shall be applied for each load type separately.

7.4.1.5 The load factors to be used for the different categories of limit states, are given in [Sec.3](#).

7.4.1.6 The material factors to be used are specified in the relevant subsection for design in this section. The characteristic strength of soil shall be assessed in accordance with [\[7.4.2\]](#).

7.4.1.7 Material factors shall be applied to soil shear strength as follows:

- for effective stress analysis, the tangent to the characteristic angle of internal friction shall be divided by the material factor γ_m , see [\[7.6.1.6\]](#).

- for total stress analysis, the characteristic undrained shear strength shall be divided by the material factor γ_m , see [7.6.1.6].

For soil resistance to axial pile load, material factors shall be applied to the characteristic resistance as described in [7.6.1.7] and [7.6.1.8].

For soil resistance to lateral pile load, material factors shall be applied to the characteristic resistance as described in [7.6.1.6].

7.4.1.8 Settlements caused by increased stresses in the soil due to structural weight shall be considered for structures with gravity-based foundations. The risk of uneven settlements should be considered in relation to the tolerable tilt of the wind turbine support structure.

Guidance note:

Special consideration should also be applied to the ULS and FLS stresses in jacket steel structures, if its pile supports settle unevenly and out-of plane (differential settlement), as e.g. occurring during a plastic behaviour of the soil around axially loaded piles.

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

7.4.2 Characteristic geotechnical parameters

7.4.2.1 When selecting a characteristic value, the following steps shall be considered:

- definition; the characteristic value shall be defined considering the design situation and the knowledge of the mobilized soil volume
- estimation; the characteristic value shall be estimated based on the amount of data available, e.g. statistical estimation according to DNVGL-RP-C207
- selection; the characteristic value shall be selected using good engineering judgement.

Guidance note:

For problems, which are governed by a local soil strength value, such as the pile tip resistance for an end-bearing pile and base shear for a monopile, a low quantile in the strength distribution should be specified as characteristic value.

For problems, which involve large soil volumes where local strength variations from point to point can be assumed to average out, such as in stability calculations for large gravity-base foundations covered by several tests locations and skin friction along an axially loaded pile, a mean value, established with confidence, of the soil strength should be specified as characteristic value. The same approach may apply for laterally loaded monopiles where a wedge type failure will occur. However, in this case, a higher confidence level than the skin friction for an axially loaded pile, should be considered due to the (often unknown) spatial variation of soil properties.

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7.4.2.2 When using statistical approaches, the designer shall assess the appropriate level of confidence to be used when establishing the characteristic geotechnical parameter.

Guidance note:

Without a detailed assessment, a confidence level of 95% should be used. For e.g. axial pile capacity, where an averaging effect can be relied upon, a confidence level of 75% may be applicable.

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7.4.2.3 For projects in which the EN 1997 series is part of the governing codes and standards, the requirements outlined by the EN 1997 series and the relevant national annex, shall be considered.

Guidance note:

As the EN 1997 series was not written explicitly for wind turbines, please consider supplementing these standards with relevant DNV GL standards as necessary.

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7.4.2.4 The characteristic strength and deformation properties shall be determined for all soil layers of importance.

7.4.2.5 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 those soil properties that govern 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.

7.4.2.6 Possible effects of installation activities on the soil properties shall be considered.

7.4.3 Loads to be applied

7.4.3.1 Additional loads resulting from imperfections and influences due to tilting of tower and foundation should be considered as outlined in e.g. [3.5.3.1] and [3.10.2.2].

7.4.4 Effects of cyclic loading

7.4.4.1 The effects of cyclic loading on the soil properties, called cyclic degradation, shall be considered in foundation design for wind turbine structures. See also DNVGL-RP-C212 Sec.10.

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

7.4.4.3 In the SLS design condition the effects of cyclic loading on the soil's shear modulus shall be corrected for as relevant when dynamic motions, settlements and permanent (long-term) rotation shall be calculated. See also [7.5.6].

7.4.4.4 The effects of cyclic loading on the ground strength and stiffness shall be addressed for ULS and SLS design condition. The following load situations will apply:

- for single storms,
- for normal operating conditions followed by a storm or an emergency shutdown,
- for several succeeding storms, and
- for any other wind and wave load condition that may influence the soil properties.
- Alternatively, one scenario may be defined, which covers the most critical ULS action on the soil surrounding the substructure, preceded by the storm or event, which leads to this very ULS action.

Guidance note:

Cyclic degradation of soil properties is the issue here, as well as rate effects.

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7.4.4.5 In seismically active areas, where the structure-foundation system may be subjected to earthquake forces, the deteriorating effects of cyclic loading on the ground properties and the effects of possible liquefaction of the soil shall be evaluated for the site-specific conditions and considered in the design where relevant. See also [7.4.5].

7.4.5 Soil-structure interaction

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

7.4.5.2 For the dynamic analysis, the distance of the natural frequency for the overall structure from the excitation frequencies is critical in avoiding resonance. In the assessment of the expected natural frequencies, a parameter study is needed for the soil parameters. In this, due consideration of the cyclic degradation effect as well as high and low estimates of the soil stiffness should be given in the dynamic analyse.

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

7.4.5.4 For analysis of the structural response to earthquake vibrations, soil motion characteristics valid at the base of the structure shall be determined. This determination shall be based on soil motion characteristics in free field and on local soil conditions using recognised methods for soil and structure interaction analysis.

7.4.5.5 For dynamic analysis of the system of wind turbine and support structure, appropriate stiffness values for the soil support of the foundation structure shall be applied. For example – in the case of pile foundations – p-y curves describing the pile-soil interaction, including appropriate initial p-y stiffness, should be applied. These requirements to appropriate representation of stiffness also apply to assessment of the natural frequency of the system of wind turbine and support structure. High or low estimates of the involved soil parameters, whatever is applicable, shall be applied for different design process targets to account for uncertainties in the estimation of an appropriate stiffness. See also [3.6].

Guidance note:

The requirement for use of appropriate values for stiffness implies that stiffness has to be represented by central estimates of its mean value when stiffness data are available. When stiffness data for a foundation are not available, appropriate stiffness value estimates may be obtained on the basis of back analyses of data for similar foundations installed in similar soil conditions.

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7.4.6 Observational method

7.4.6.1 Usually, a design process is based on site actions and material/soil resistances both known by long experience with a certain probability. The whole design process is based on these values.

If actions or resistances or both are less than usually known, a design may be based on conservative values or on the observational method. This method has the following elements:

- 1) Design assumptions have to be deliberately chosen, i.e. actions and material/soil resistances, even if their underlying data basis is not validated to the same degree as customary in industry practice.
- 2) A design is worked out on basis of these estimated assumptions by a standard design process.
- 3) A monitoring concept has to be worked out which is able to test the design assumptions, both for loads and material/soil resistance. The critical values to be measured by the monitoring have to be determined

- in advance as well as their critical trigger values, beyond which human induced action (repair/mitigation) is required. The concept has to be chosen in such a way, that a structural repair can be accomplished before the structure is damaged beyond repair. The monitoring shall be physically possible.
- 4) A repair/mitigation concept has to be worked out to adjust the structure in case the monitoring shows that the design assumptions were not conservative enough, i.e. if the monitored values exceed the trigger values. The planned repair time has to be chosen in a way, that the structure can at least withstand those ULS loads which have a recurrence period of the repair time. In this case, at least the material factors of the accidental load scenario under the measured loads have to be attained. For the fatigue loads, equivalent assumptions have to be fed into the design. The repair shall be physically possible.
 - 5) Steps 2 to 4 may be repeated several times before a satisfying design solution is attained. Especially 4 can only be proven after the realisation of the design on site and the first monitoring results are collected.
 - 6) The design is realized on site.
 - 7) Monitoring and, if required, repair is carried out.

Ideally, loads on the structure can be adjusted by human induced action in case the monitoring reveals a condition outside the design assumptions.

Guidance note:

The rules for the design of scour protection (see [Sec.8](#)) may be seen as an example of the observational method. Another application of this method may be seen in the case of a pile design based on pile tests. If the pile test does not show the desired results, the pile bearing capacity has to be improved afterwards, by suitable means.

Due to inherent risks of this trial and error method, it should only be used if the standard design process is not possible.

Please also refer to EN 1997-1.

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7.5 Design of gravity-based foundations

7.5.1 General

7.5.1.1 Gravity-based foundations are characterized as foundations with relatively small penetration into the soil compared to the width of the foundation bases and relying predominantly on compressive contact with the supporting soil.

7.5.1.2 Failure modes within the categories of limit states ULS and ALS shall be considered as described in [\[7.5.2\]](#). Material factors for ULS are given in the relevant sections and material factor for ALS shall be taken as $\gamma_m = 1.0$.

7.5.1.3 Failure modes within the SLS, i.e. settlements and displacements, shall be considered as described in [\[7.5.3\]](#) using a material factor of $\gamma_m = 1.0$.

7.5.2 Stability of foundations

7.5.2.1 The risk of shear failure below the base of the structure shall be investigated for all gravity-based 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.

Guidance note:

If soil layers of sufficient strength are not found at some depth below the soil surface, a soil replacement or foundation skirts may be required in order to ensure the suitability of the foundation. Properly stiffened soil skirts transfer the effective interface between foundation loads and soil down to the level of the skirt's tip, thus increasing their capacity.

Offshore, in some cases skirts will be used to protect the foundation against scouring.

For gravity-based structures equipped with skirts which penetrate the seabed, the theoretical foundation base should be assumed to be at the skirt tip level.

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

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

- effective stress stability analysis
- total stress stability analysis.

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

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

7.5.2.6 Both effective stress and total stress analysis methods shall be based on laboratory shear strength with pore pressure measurements included where feasible.

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

Guidance note:

Gravity-based foundations of wind turbines usually have relatively small areas, such that bearing capacity formulae for idealised conditions will normally suffice and be acceptable for design.

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7.5.2.8 For structures where skirts, dowels or similar foundation members transfer loads to the foundation soil, the contributions of these members to the bearing capacity and lateral resistance may be accounted for as relevant. The feasibility of penetrating the skirts shall be adequately documented. Please also consider [7.5.4.3].

7.5.2.9 Foundation stability shall be analysed in the ULS by application of the following material factors to the characteristic soil shear strength parameters:

$\gamma_m = 1.15$ for effective stress analysis

$\gamma_m = 1.25$ for total stress analysis.

7.5.2.10 Effects of cyclic loading shall be included by applying load factors in accordance with [7.4.1.4].

7.5.2.11 In an effective stress analysis, evaluation of pore pressures should include:

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

7.5.2.12 The safety against overturning and sliding shall be investigated in the ULS and in the ALS.

7.5.3 Settlements and displacements

7.5.3.1 For SLS design conditions, analyses of initial and long term settlements and displacements are, in general, to include calculations of:

- initial consolidation and secondary settlements
- even and differential settlements
- permanent horizontal displacements
- dynamic motions.

Please also refer to [7.5.5].

7.5.3.2 Displacements of the structure, as well as of its foundation soils, shall be evaluated to provide the basis for design of cables and J-tubes and other members connected to the structure which are penetrating the soil surface or resting on the surface.

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

7.5.4 Soil reactions on foundation structure

7.5.4.1 The reactions from the soils shall be accounted for in the design of the supported structure for all design conditions.

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

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

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

7.5.5 Gapping between foundation and soil

7.5.5.1 The load conditions on the foundation structure may be of such a nature, that there is no contact pressure between foundation and soil in a part of the foundation area. Such a state is called gapping. If such a state occurs repeatedly due to cyclic loads, it is called gapping cycles.

7.5.5.2 The foundation shall not experience negative effects due to repeated gapping cycles. The stress state in the interface has to be such, that the shape of the soil plane under the foundation remains stable over the whole operational life of the structure above it. Neither differential settlements nor changes in the shape of the soil under the foundation shall occur in such magnitude, that the soil in the interface is permanently changed to a degree by which the operation of the structure above it is endangered.

In the following paragraphs a set of empiric rules are described, whose applicability has been proven for wind turbines and their typical loads over a long time by practical experience. Other methods may be applicable.

7.5.5.3 For gravity-based foundations no gapping should occur under the SLS load case LDD 10^{-2} as defined in [3.10]. This shall be shown by an analytical calculation, treating the soil as an ideal elastic half space.

7.5.5.4 For the characteristic extreme load as defined in [3.10] gapping is only permitted up to the center of gravity of the bottom area of the foundation.

7.5.5.5 Exceptions from [7.5.6], [7.5.5.3] are allowable if it is verified by calculation or testing that water pressure in the interface does not lead to soil shape change below the foundation in case of gapping.

It has to be observed that the effective soil resistance may be reduced due to cyclic loading or hydraulic gradients, induced for example by foundation rocking.

The loss in contact area has to be taken into account in the rotational foundation stiffness calculations.

7.5.6 Soil modelling for dynamic analysis

7.5.6.1 Dynamic loads due to wind, wind turbine operation, waves for offshore wind turbines, earthquake etc. may significantly influence the integrity of the foundation. Their effects on the foundation behaviour have to be evaluated.

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

Guidance note:

When the soil conditions are fairly homogeneous and an equivalent shear modulus G can be determined, representative for the participating soil volume as well as for the prevailing strain level in the soil, then the foundation stiffnesses may be determined based on formulae from elastic theory, see DNVGL-RP-C212.

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7.5.6.3 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 high and low estimates on shear moduli and damping ratios of the soil. Both internal soil damping and radiation damping shall be considered.

7.5.7 Underbase grouting for offshore foundations

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

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

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

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

7.5.8 Installation of offshore foundations

7.5.8.1 Careful planning is necessary in order to ensure a proper installation of the foundation base.

7.5.8.2 When skirts and other installation aids (dowels etc.) have to penetrate into the sea bottom, a penetration analysis has to be performed using the soil characteristics of the relevant soil layers.

7.5.8.3 The requirements on ballasting facilities have to be investigated in order to ensure a well-balanced seating of the foundation base without excessive disturbance of the supporting soil.

7.6 Design of pile foundations

7.6.1 General

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

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

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

7.6.1.3 The data bases of existing methods for calculation of soil resistance to axial and lateral pile loads are often not covering all conditions of relevance for offshore piles. This in particular relates to size of piles, soil shear strength and type of load. When determining the 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.

7.6.1.4 It shall be demonstrated that the selected solution for the pile foundation is feasible with respect to installation of the piles. For driven piles, this may be achieved by a driveability study or an equivalent analysis.

Guidance note:

For evaluation of pile drivability, it is important to apply well documented methods or, alternatively, reverse analyses from similar piles in similar soil conditions.

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7.6.1.5 Structures with piled foundations shall be assessed with respect to stability for both operation and temporary design conditions, e.g. prior to and during installation of the piles.

Guidance note:

For drilled piles, it is important to check the stability of the drilled hole in the temporary phase before the pile is installed in the hole.

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7.6.1.6 Unless otherwise specified, the following material factors γ_m shall be applied to the characteristic soil strength parameters for determination of design soil resistance against lateral loading of piles in the ULS and the SLS:

Table 7-1 Material factors for pile foundations

Type of geotechnical analysis	Limit state	
	ULS	SLS

	γ_m	γ_m
Effective stress analysis	1.15	1.0
Total stress analysis	1.25	1.0

7.6.1.7 If the axial pile loads are determined by a calculation of the complete foundation structure with the target, that no single pile reaches its ultimate capacity, a material factor $\gamma_m = 1.25$ shall be applied to all characteristic values of soil resistance to determine its design capacity. This material factor is valid for the characteristic limit skin friction in tension and compression and for the characteristic tip resistance.

Guidance note:

The above material factor $\gamma_m = 1.25$ for axially loaded piles should be applied to jacket pile foundations which still have, apart from the pure axial pile bearing capacity, a possibility to activate additional bearing capacity working against the overall failure of the wind turbine support structure. This means, a load redistribution is still possible after the maximum soil bearing strength for the most critical pile is reached. If the most critical pile is loaded in tension, usually a load redistribution is not possible.

The design pile loads should be determined from structural analyses in which the pile foundation is modelled either with an adequate equivalent elastic stiffness or with non-linear models that reflect the true non-linear stress-strain properties of the soil in conjunction with the characteristic ground strength.

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7.6.1.8 If the ultimate resistance of the foundation pile system is analyzed by modelling the soil with its design strength and allowing full plastic redistribution until a single global foundation failure chain is reached, the material factor for skin friction in compression and tip resistance shall be increased to $\gamma_m = 1.4$. If piles loaded in tension are critical for the overall strength of the substructure the material factor shall be $\gamma_m = 1.5$.

7.6.1.9 Additionally, the consequences of pile load redistribution on the substructure have to be shown for ULS and FLS loads. This comprises at least the consideration of the differential settlements accompanying the plastic bearing behaviour of the piles and its implications for substructure design details as tube joints etc.

7.6.1.10 For individual piles in a group lower material factors may be accepted, as long as the pile group as a whole is designed with the required material factor. A pile group in this context is a number of neighboring piles, which are rigidly joined by a connecting element, which limits the differential deformations of all members of the pile group to a negligible amount. For example a pile group shall not include more piles than those supporting one specific jacket leg.

Guidance note:

For cohesionless soils, there is a difference between the skin friction in compression and in tension, which has to be taken into account in determining their values. Care has to be applied when comparing different calculation methods for pile bearing capacities.

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7.6.1.11 For drilled piles, where the installation method is liable to negatively change the friction between outer pile surface and the in-situ soil (e.g. by using a non-set bentonite to stabilize the borehole), the assumptions made for the limit skin friction in design shall be verified during the installation or by test results of comparable pile in comparable soils. This may also be accomplished by for example dynamic pile testing, if site conditions are favorable for this type of testing.

Guidance note:

The drilling mud (e.g. bentonite) which is used during the drilling of the hole for the pile influences the adhesion between the pile and the soil and thereby also the limit skin friction.

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7.6.1.12 Laterally loaded piles may be analyzed on the basis of appropriate characteristic stress-strain curves for soil and pile. The pile deflections induced by the combination of lateral and axial loading may be so large that inelastic behaviour of the soil takes place.

7.6.1.13 The lateral resistance of a pile or a pile group may in the ULS be based on the theory of plasticity provided that the characteristic resistance is in accordance with recognized plastic theorems so as to avoid non-conservative estimates of the safety. The calculations are then to be based on the assumption that the lateral deformations of the pile are sufficiently large to plastify the soil completely.

7.6.1.14 For analysis of pile stresses and lateral pile head displacements, the lateral pile resistance shall be modelled using characteristic soil strength parameters, with the material factor for soil strength equal to $\gamma_m=1.0$. Non-linear response of soil shall be accounted for, including the effects of cyclic loading.

7.6.1.15 For design of the serviceability limit states (especially for lateral loads), characteristic values shall be used for the soil strength. Characteristic values shall be used for the loads. The loading shall be representative for loads that will cause permanent deformations of the soil in the long term, and which in turn will lead to permanent deformations of the pile foundation, e.g. a permanent accumulated tilt of the support structure or the pile head. For this purpose, the behaviour of the soil under cyclic loading needs to be represented in such a manner that the permanent cumulative deformations in the soil are appropriately calculated as a function of the number of cycles at each load amplitude in the applied history of SLS loads.

7.6.1.16 For design in the serviceability limit states, it shall be ensured that deformation tolerances are not exceeded.

Guidance note:

Lateral deformation tolerances are usually given in the design basis and they are often specified in terms of maximum allowable rotations of the support structure and maximum allowable horizontal displacements of the pile heads.

Separate tolerances may be specified for the support structure and piles for the situation immediately after completion of the installation and for the permanent cumulative damages owing to the history of SLS loads applied to the structure and foundation throughout the design life.

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7.6.2 Design criteria for laterally loaded piles

7.6.2.1 For geotechnical design of lateral loaded piles, both the ultimate limit states and the serviceability limit states shall be considered.

7.6.2.2 When pile penetrations are governed by lateral pile resistance, the design resistance shall be checked with respect to the ULS. For the ULS, material factors as prescribed in [7.6.1.6] shall be used.

7.6.2.3 For design in the ultimate limit state, design resistance values should be used, defined as the characteristic soil shear strength values divided by the specified material factor.

7.6.2.4 Design loads shall be used for the loads, each design load being defined as the characteristic load multiplied by the relevant specified load factor. The loads shall be representative of the extreme load conditions.

7.6.2.5 For combined lateral loading and moment loading in the ULS, sufficient pile capacity against this loading shall be ensured. The pile capacity is formed by lateral pile resistance. Verification of sufficient pile capacity implies that the following two requirements shall be fulfilled:

- 1) The theoretical design total lateral pile resistance, which is found by vectorial integration of the design lateral resistance over the length of the pile, shall not be less than the design lateral load applied at the pile head.

- 2) The lateral displacement at the pile head shall not exceed some specified limit which is defined in the corresponding design basis for example. The lateral displacement shall be calculated for the design lateral load and moment in conjunction with characteristic values of the soil resistance and soil stiffness.

Requirement (1) is the conventional design rule, which is based on full plastification of the soil. Requirement (2) is a necessary additional requirement, because lateral soil resistance cannot be mobilised locally in zones near points along the pile where the direction of the lateral pile deflection is reversed, i.e. the soil in these zones will not be fully plastified, regardless of how much the pile head deflects laterally.

Guidance note:

Sufficient pile capacity against combined lateral loading and moment loading may be ensured by means of a so-called single pile analysis in which the pile is discretized into a number of structural elements, interconnected by nodal points, and with soil support springs in terms of p-y and t-z curves attached at these nodal points. Lateral forces and overturning moments are applied to the pile head. Also axial forces acting at the pile head need to be included, because they may contribute to the bending moment and the mobilization of lateral soil resistance owing to second-order effects.

It is important that the p-y curves used for representation of the lateral support in this analysis account for the cyclic degradation effects in the lateral resistance and stiffness.

The acceptance criterion for sufficient lateral pile resistance needs to be a criterion on displacement, see requirement (2). A criterion on the lateral deflection of the pile head or a criterion on the rotation of the pile head about a horizontal axis will be practical. When particularly conservative assumptions have been made for the lateral soil resistance, requirement (2) may be waived.

It will usually not suffice to ensure that the lateral design load at the pile head does not exceed the design total lateral resistance that is theoretically available and which can be obtained from the single-pile analysis. This is so because long before the total available lateral resistance becomes mobilised by mobilisation of all lateral soil resistance along the pile, excessive (and unacceptable) lateral pile displacements will take place at the pile head.

When carrying out a single-pile analysis, it is recommended to pay attention to the lateral pile head displacements that result from the single-pile analysis and make sure that they do not become too large, e.g. by following the predicted pile head displacement as function of the pile length and making sure that the design is on the flat part of the corresponding displacement-length curve.

It is also recommended to make sure that the soil zones along the pile, which are plastified for the lateral ULS loads, are not too extensive.

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7.6.2.6 Use of p-y curves for design of piles with diameters of more than 1.0 m (for example monopiles) is recommended to be validated for such use, e.g. by means of FE analysis.

7.6.2.7 For design in the serviceability limit states, it shall be ensured that deformation tolerances are not exceeded. The deformation tolerances refer to permanent deformations.

Guidance note:

Deformation tolerances are usually given in the design basis and they are often specified in terms of maximum allowable rotations of the pile head in a vertical plane. The pile head is usually defined to be at the soil surface. The deformation tolerances are typically derived from visual requirements and requirements for the operation of the wind turbine. The deformation tolerances should therefore always be clarified with the wind turbine manufacturer.

Usually, an installation tolerance is specified in the Design Basis due to functional requirements.

For offshore monopiles, in addition, another tolerance is usually specified which is an upper limit for the accumulated permanent rotation of the pile head due to the history of SLS loads applied to the monopile throughout the design life. The accumulated permanent rotation subject to meeting this tolerance usually results from permanent accumulated soil deformations caused by cyclic wave and wind loads about a non-zero mean.

In some cases, an installation tolerance is specified together with a tolerance for the total rotation owing to installation and permanent accumulated deformations. This is usually expressed as a requirement to the rotation or tilt of the pile at the pile head, where the pile head is defined as the position along the pile in level with the seabed. If, for example, the tolerance for the total rotation at seabed is 0.5° and the installation tolerance at seabed is 0.25°, then the limit for the permanent accumulated rotation becomes 0.25° at seabed.

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7.6.3 Design criteria axially loaded piles

7.6.3.1 For geotechnical design of axially loaded piles, both the ultimate limit states and the serviceability limit states shall be considered.

7.6.3.2 Sufficient axial pile capacity in the ULS should be ensured for each single pile.

Guidance note:

The verification of sufficient axial capacity of the individual piles may be performed by means of an integrated analysis of the entire support structure and its foundation piles, subject to the relevant design loads.

In such an analysis, the piles are discretised into a number of structural elements, interconnected by nodal points, and with soil support springs in terms of p-y and t-z curves attached at these nodal points to represent lateral and axial load-displacement relationships, respectively.

The t-z curves depend on the unit skin friction. For clay, the unit skin friction is a function of the undrained shear strength. For sand, the unit skin friction is a function of the relative density. In both cases, the unit skin friction may be determined as specified in DNVGL-RP-C212 and ISO 19902.

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7.6.3.3 Cyclic loads, especially the cyclic loads of wind turbines, as well as the combination of the fatigue loads with the maximum ULS load have to be considered in design.

Guidance note:

It is important to consider the effects of the cyclic loading on the unit skin friction. The degradation of the unit skin friction should be determined for the relevant prevailing load intensities before the t-z curves are generated.

The effects of cyclic loading are most significant for piles in cohesive soils, in cemented calcareous soils and in fine-grained cohesionless soils (silt), whereas these effects are much less significant in medium to coarsely grained cohesionless soils. In general, a detailed calculation of the effects of cyclic loads induced by standard wind turbines may be omitted if it is shown that no tensile forces occur for axially loaded piles under the SLS load case LDD 10^{-2} as defined in [3.10].

Regarding the maximum ULS load scenario for offshore wind turbines for some specific national requirements (e.g. according to BSH 7005 in Germany), the fatigue loads of a 35 hours storm combined with the maximum ULS load (which is embedded in the 35 hours storm) may be chosen as the most critical load combination, see also BSH 7005, App.3. The fatigue loads should have a $\gamma_f = 1.0$ and the ULS load its corresponding ULS-specific load factor γ_f . For the fatigue loads over the operational lifetime, a separate analysis has to be set up.

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7.6.4 Pile groups

7.6.4.1 For foundations consisting of pile groups, i.e. clusters of two or more piles spaced closely together, pile group effects need to be considered when the axial and lateral resistance of the piles shall be evaluated. For a pile spacing less than eightfold the pile diameter group effects shall be evaluated.

Guidance note:

When piles are closely spaced, the resistance of the piles as a group may be less than the sum of the individual pile capacities, both laterally and axially, and the lateral and axial resistances of the p-y and t-z curves should be adjusted accordingly.

When piles are closely spaced, the load transferred from each pile to its surrounding soil leads to displacements of the soil that support the other piles, and the behaviour of the piles as a group may be softer than if the piles were considered to have supports which were not displaced by influence from the neighbouring piles. This effect may in principle be accounted for by elastic half-space solutions for displacements in a soil volume due to applied point loads.

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7.6.4.2 For axial loading, sufficient axial pile capacity in the ULS shall be ensured for each single pile. For combined lateral loading and moment loading, sufficient pile capacity against this loading in the ULS shall be ensured for each single pile.

7.6.5 Design of offshore piles subjected to scour

7.6.5.1 Effects of scour shall be accounted for. Scour will lead to complete loss of lateral and axial resistance down to the depth of scour below the original seabed. Both general scour and local scour shall be considered. The p-y and t-z curves shall be constructed with due consideration of the effects of scour.

In the case of general scour, which is characterised by a general erosion and removal of soil over a large area, all p-y and t-z curves shall be generated on the basis of a modified seabed level which shall be taken as the original seabed level lowered by a height equal to the depth of the general scour.

General scour reduces the effective overburden. This has an impact on the lateral and axial pile resistances in cohesionless soils. This also has an impact on the depth of transition between shallow and deep ultimate lateral resistances for piles in cohesive soils.

In the case of local scour, which is characterised by erosion and removal of soil only locally around each pile, the p-y and t-z curves should be generated with due account for the depth of the scour hole as well as for the lateral extent of the scour hole. The scour-hole slope and the lateral extent of the scour hole can be estimated based on the soil type and the soil strength. Over the depth of the scour hole below the original seabed level, no soil resistance and thus no p-y or t-z curves shall be applied.

The risk of scour shall be conservatively considered for eigenfrequency calculation, see also [3.4].

7.7 Offshore suction bucket foundations

7.7.1 General

7.7.1.1 All suction installed foundations, such as suction anchors, suction caissons, suction piles, suction buckets and suction cans are hereafter all referred to as suction bucket foundations. For the description of the concept please refer to [7.2.8].

7.7.1.2 Based on the difference between the loading conditions on the foundation, and on the superstructure connected to it, the suction buckets can be divided into two main groups:

- monobuckets
- buckets for jacket structures.

7.7.1.3 The suction bucket foundation concept is not covered by the available codes, thus a case-to-case evaluation is required. Thorough literature study should be performed prior to the design, and information based on former experiences should be exploited.

7.7.2 Installation

7.7.2.1 Feasibility of installation may be evaluated using recognized methods for skirt penetration as for example found in DNVGL-RP-C212.

7.7.2.2 The suction bucket structure should be designed against the applied underbase suction pressure and the forced deflection during installation, due to the tangential and meridional imperfections of the suction bucket structure. Buckling analyses of the shell and top plate shall be carried out, and construction tolerances and possible soil unevenness shall be considered.

Guidance note:

The possible formation of flow channels at the vertical wall may significantly reduce the skirt resistance.

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7.7.3 In-place conditions

7.7.3.1 It is important to understand the drainage mechanisms in the soil stratigraphy, since it has an impact on the design. There are different effects of a pressure gradient for drained, partially drained and undrained soil conditions. When the drainage mechanisms cannot be determined with good confidence, conservative assumptions have to be made.

Guidance note:

The permeability of the layers, both vertical and horizontal, should be determined for the soil types encountered within the confined space of the bucket and at the skirt tip. This helps evaluating the pressure distribution from the bucket lid to the seabed.

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7.7.3.2 Soil strength degradation from cyclic loads shall be considered. There are potential favourable and unfavourable consequences of the rate effect found from cyclic tests. Therefore the loading regime should be well understood. The environmental conditions can also entail high pullout loads, which should also be accounted for.

Guidance note 1:

High level (high strain) ULS loading has the potential to reduce strength. Low level (low strain) FLS loading has the potential to result in a strength increase, due to the self-healing nature of the soil, or any possible pre-shearing. The coefficients of consolidation of the soils are in connection with the loading rate, and should also be evaluated.

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Guidance note 2:

When assessing the cyclic effects of environmental loading both pore pressure build up and dissipation should be considered. Pore pressure dissipation may lead to loss of tension capacity due to water flow into the foundation bucket.

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7.7.3.3 There are various geotechnical phenomena which can pose at risk or limit the design of the suction bucket structure, and which therefore need to be addressed. These are including, but not limited to scour, the potential for plug heave or plug lift, cavitation, formation of flow channels adjacent to the vertical skirt or gapping.

7.7.4 Soil reactions on foundation structure

7.7.4.1 All the potential effects of the soil on the foundation structure shall be duly evaluated and accounted for in determining the load cases and load combinations for the structural analyses.

7.7.4.2 Finite element analysis or scale model tests are recommended to better assess the possible failure modes, drainage mechanisms, effective stresses and the effects of high or low level cyclic loading.

7.8 Stability of soil surface

7.8.1 Slope stability

7.8.1.1 The risk of slope failure shall be evaluated. For example:

- natural slopes
- slopes developed during and after installation of the structure
- future anticipated changes of existing slopes
- effect of continuous mudflows
- wave induced movements (both soil and structure) for offshore wind turbines.

The effect of wave loads at the sea bottom shall be included in the evaluation for offshore support structures when such loads are unfavourable.

7.8.1.2 In a seismically active region, the effects of earthquakes shall be considered.

7.8.1.3 The safety against slope failure for ULS design shall be analyzed using material factors (γ_m):

$\gamma_m = 1.15$ for effective stress analysis

$\gamma_m = 1.25$ for total stress analysis.

7.8.2 Hydraulic stability

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

7.8.2.2 The hydraulic stability shall be analysed for:

- softening of the soil and consequent reduction of bearing capacity due to flow induced hydraulic gradients
- 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
- wind and wave loading.

7.8.2.3 When erosion is likely to reduce the effective foundation area, measures shall be taken to prevent, control and/or monitor such erosion, as relevant.

7.9 Removal of foundations

7.9.1 General

7.9.1.1 In case a removal is agreed between owner/operator, designer and the relevant administration, reasonable assumptions and supporting investigations shall be carried out regarding potential options for:

- method and phases of removal
- environmental conditions
- risks involved
- necessary equipment to be provided during operations
- structural arrangements and mechanical devices (pipes, fittings etc.) to be provided already during construction phase, and measures required to ensure that removal operations will be possible at the expected time.

7.9.1.2 If removal is anticipated, this should be assessed by analyses of the soil reactions generated during the removal procedure. This should be carried out in order to ensure that the resistance can be overcome with the means available. The analysis shall be based on upper bound soil parameters.

Guidance note:

Regarding skirted foundations, suction forces tend to develop at the foundation base and the tips of skirts. These forces may be overcome by sustained uplift forces or by introducing water into the confined base compartments to relieve the suction.

Setup effects, consolidation effects, uneven separation from soil surface, possible drop-off of soil or underbase grout, weights of accumulated debris and marine growth should be considered.

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7.9.1.3 Suction installed foundations can be removed by reversing the installation process and applying overpressure in the bucket instead of underpressure.

7.10 Power cable trenches

Different requirements apply to the routes for infield cables, and subsea cables (export and array cables) regarding their horizontal cover and penetration depth. Seabed hardness and trenchability is of particular interest with regard to cable burial depth. The state-of-the-art cable trenching methods are ploughing, jetting and cutting. For further details, reference is made to DNVGL-ST-0359 and DNVGL-RP-0360.

SECTION 8 SCOUR AND SCOUR PREVENTION FOR OFFSHORE STRUCTURES

8.1 General

8.1.1 Introduction

8.1.1.1 When a structure is placed on the seabed, the near bed flow may undergo substantial local changes. Those flow changes may, in case of an erodible seabed, cause local erosion and local deposition, resulting in scour development. Such scour may reduce the stability of the structure and has to be considered in the design. Also global scour and general seabed level changes have to be included in the design.

8.1.1.2 Sea bed Bathymetry (e.g. sand banks, sanddunes, sandbars etc.).

8.1.1.3 For offshore environments the risk for scour development around the substructure shall be evaluated.

8.1.1.4 If the substructure is placed without scour protection the scour has to be considered in the design.

8.1.1.5 If the substructure is placed with scour protection, the scour protection stability has to be documented.

8.1.2 Non protected structures

8.1.2.1 App.D presents simple formula for scour depths around a pile placed in non-cohesive sediment.

8.1.2.2 Piles placed cohesive sediment (such as silt or clay) App.D may not be applicable.

8.1.2.3 For other structures (for example GBS, jackets structures etc.), or structures places on cohesive sediment or in a wave dominated environment the scour development has to be predetermined by full scale or model scale experiences of comparable sites and geometries. Sufficient safety has to be added depended on the uncertainties (environmental conditions, soil conditions etc.).

8.1.2.4 Potential long-term general seabed level changes shall be considered.

8.1.2.5 The support structure has to be designed for both upper and lower seabed levels in combination with upper and lower scour depths.

8.1.2.6 If the design scour depth may be exceeded the scour and general seabed level at site have to be monitored with regular intervals and after severe storms. If the measured scour depths/seabed levels are below the design levels, and if structural integrity cannot otherwise be documented a scour protection has to be installed.

8.1.3 Scour protection

If scour protection is installed the scour protection shall be designed to provide protection of the seabed against local scour. Riprap scour protection shall be designed to be hydraulically stable against excessive surface erosion of the scour protection itself and excessive transportation of soil particles from the underlying natural soil. Edge scour, global scour, seabed lowering and local scour protection sinking may occur also for scour protected structures and shall be considered in the design for the overall integrity of the foundation structure.



Guidance note:

Care should be taken if a structure is placed in an area with large bathymetrical changes, for example the scour protection have to accommodate the lowering of the surrounding seabed.

In cases where a scour protection is in place at a foundation structure, the weight of the scour protection may be taken into account in the design of the foundation structure, if it can be shown that the scour protection are fully stable in the entire lifetime. Often it is too costly to install a scour protection that can be considered to be 100% stable, in such cases the scour protection and general seabed level at site have to be monitored with regular intervals and after severe storms. If the measured seabed levels are within the design levels and no damage on the scour protection has occurred, no further action is required.

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

If the measured seabed levels are outside the design seabed levels or the scour protection is damaged proper action shall be taken.

SECTION 9 IN-SERVICE INSPECTION, MAINTENANCE AND MONITORING

9.1 Introduction

9.1.1 General

9.1.1.1 A wind turbine support structure is typically planned for a design lifetime of 20 to 30 years. In order to sustain the impact from power production and from the environment, adequate inspection and maintenance have to be carried out for the support structures. This section provides requirements and recommendations for inspection and maintenance of wind turbines support structures.

9.1.1.2 The design of the wind turbine support structures shall take into account the practicability of carrying out inspections of relevant structures.

9.1.1.3 A program for inspection of the wind turbine support structures in a wind farm should be defined and implemented. In general, the design of a program for inspection shall be based on a systematic assessment of potential failures. Programs for in-service inspections of wind turbine support structures depend on the number of structures in a wind farm in question. They also depend on the design and the specific environmental conditions etc.

9.1.1.4 Where inspection is not practicable, the structures shall be designed and constructed so that adequate durability for the entire operating life of the installation is assured.

9.1.1.5 For single wind turbines and for wind farms comprising only a few wind turbines, it may be feasible to define rigid inspection programs for the support structures with requirements for annual inspections and other periodical surveys which cover all structures in the wind farm. For large numbers of wind turbine structures in large wind farms, such rigid inspection programs can be far too comprehensive to carry out, and inspection programs defined from risk-based inspection planning are recommended. In wind farms with many series manufactured identical or almost identical structures, it suffices to carry out inspections on a few representative structures only.

9.1.1.6 All information required for utilization and maintenance should be at the disposal of whoever has responsibility for the entire structure.

9.2 Periodical inspections – general

9.2.1 General

9.2.1.1 The provisions of this subsection apply to wind turbine support structures for which periodical inspections are chosen as the approach to in-service inspection.

9.2.1.2 The periodical inspection can consist of three different levels of inspection:

- general visual inspection
- close visual inspection
- non-destructive examination.

9.2.1.3 For offshore structures general visual underwater inspections can be carried out using a remotely operated vehicle (ROV), whereas close visual underwater inspections could require inspections carried out by a diver.

9.2.2 Preparation for periodical inspections

9.2.2.1 Long term inspection program for the wind farm shall be prepared, in which all disciplines and systems to be covered by the program are specified. In this program, inspection coverage over a five-year period should be specified in order to ensure that all essential parts of the support structure will be covered by annual inspections over the five-year period.

9.2.2.2 The periodical inspections should be carried out with a scope of work necessary to provide evidence as to whether the inspected structures or structural components continue to comply with the design assumptions.

9.2.2.3 The scope of work for an inspection shall always contain a sufficient number of elements and also highlight any findings or deviations reported during previous inspections which have not been dealt with.

Guidance note:

The inspection will typically consist of an office part and an on-site part.

The office part typically includes:

- follow up on outstanding issues from the previous inspection
- revision of inspection procedures
- revision of maintenance documentation interview with discipline engineers, including presentation/clarification of any comments deduced during review of procedures
- review of maintenance history
- preparation of the on-site program, based on findings from the office part and systems selected from the long term inspection program.

The on-site inspection typically includes test and inspections on site as well as an assessment of the findings in order to distinguish between random failures and systematic failures.

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9.2.3 Interval between inspections

9.2.3.1 The interval between inspections of critical items should not exceed one year. For less critical items longer intervals are acceptable. The entire wind farm should be inspected at least once during a five-year period. Inspection intervals for subsequent inspections should be modified based on findings. Critical items are assumed to be specified for the specific project in question.

9.2.4 Inspection results

9.2.4.1 The results of the periodical inspections shall be assessed and remedial actions taken, if necessary. Inspection results and possible remedial actions shall be documented.

9.2.5 Reporting

9.2.5.1 The inspection shall be reported. The inspection report shall give reference to the basis for the inspection such as design assumptions, national regulations, rules and inspection programs, instructions to surveyors and procedures. It shall be objective, have sufficient content to justify its conclusions and should include good quality sketches and/or photographs as considered appropriate.

9.3 Periodical inspections – steel structures

9.3.1 General

9.3.1.1 The provisions of this subsection apply to steel support structures for which periodical inspections are chosen as the approach to in-service inspection.

9.3.1.2 The wind turbine manufacturer's service manual for the wind turbines shall be consulted for its specification of requirements for inspections of the steel wind turbine towers and for information about equipment which may interfere with the support structure.

9.3.2 Scope for inspection of steel structures

9.3.2.1 Examples of issues and items to be covered by the inspection are:

- fatigue cracks
- dents
- deformations
- bolt pre-tension
- corrosion protection systems
- anchor points for fall protection
- lifting appliances
- marine growth for offshore structures.

9.3.2.2 Inspection for fatigue cracks at least every year may be waived depending on which design philosophy has been used for the structural detail in question: When the fatigue design of the structural detail has been carried out by use of safety factors corresponding to an assumption of no access for inspection according to [Table 4-18](#), then there is no need to inspect for fatigue cracks and inspection for fatigue cracks may be waived. When smaller safety factors have been used for the fatigue design, inspections need to be carried out. The inspection interval depends on the structural detail in question and the inspection method and may be determined based on the magnitude of the safety factor applied in design. In general, the smaller the safety factor, the shorter is the interval between consecutive inspections.

Guidance note:

Provided a reliable inspection, such as an inspection by eddy current or a magnetic particle inspection, is carried out after a good cleaning of the hot spot area, the interval between consecutive inspections can be calculated from the safety level expressed in terms of the material design fatigue factor DFF as follows:

Inspection interval = Calculated fatigue life · DFF/3.0

This implies the following requirements to inspection:

- DFF = 3.0 No check for fatigue cracks is needed, corresponding to an assumption of no access to the structural detail.
- DFF = 2.0 Checks for fatigue cracks needed every 13 years if the calculated fatigue life is 20 years. This will result in the same safety level as that achieved for DFF = 3.0 without inspections.
- DFF = 1.0 Checks for fatigue cracks needed every 7 years if the calculated fatigue life is 20 years. This will result in the same safety level as that achieved for DFF = 3.0 without inspections.

For special cases other methods such as close visual inspections (CVI) could be used in atmospheric zone to justify the reduction of safety by choice of DFF = 2.0. This requires that the detail in question has the ability to develop a crack which is detectable by CVI before the crack leads to an unstable fracture.

The close visual inspection has to be performed according to an inspection plan. The inspection plan should identify fatigue-critical details derived from the design calculations and also the inspection interval as a minimum.

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

9.3.2.3 If cathodic protection is applied the protection potential shall be measured and fulfil minimum requirements.

9.4 Periodical inspections – concrete structures

9.4.1 General

The provisions of this subsection apply to concrete support structures for which periodical inspections are chosen as the approach to in-service inspection.

9.4.2 Scope for inspection of concrete structures

9.4.2.1 Concrete surfaces shall be inspected for cracks, abrasion, spalling and any signs of corrosion of the steel reinforcement and embedments.

9.4.2.2 Concrete structures shall be inspected especially where repairs have been carried out previously. Cleaning of the surface may be necessary.

9.4.2.3 Offshore concrete structures shall be inspected especially in the splash zone and in areas exposed to sea ice.

9.4.2.4 During the operating life of the wind turbine, anchor bolt connections as well as post-tensioning systems shall be inspected and tested as part of maintenance.

9.4.2.5 For anchor bolts an interval for regular close visual inspections and looseness checks during the life cycle of the wind turbine shall be specified. The intention of looseness checks is to detect if bolts have lost the prestress, have failed, or have not been tightened.

9.4.2.6 For tightening of anchor bolts torque-controlled tensioning shall be avoided.

9.4.2.7 The result of the inspection shall be reported in the inspection report.

9.5 Periodical inspections – grouted connections

Periodic monitoring is recommended for grouted connections. In case movements are observed measurements and periodic monitoring have to be launched.


9.6 Periodical inspections – scour

For offshore support structures periodic inspection of the scour protection may be required depending on the chosen design philosophy.

9.7 Inspections according to risk-based inspection plan

9.7.1 General

9.7.1.1 The provisions of this subsection apply to wind turbine support structures for which inspections of a few representative structures in a large wind farm and according to a risk-based inspection plan are chosen as the approach to in-service inspection.



9.7.1.2 For inspection of a few representative structures in a large wind farm, it is recommended as a minimum to survey one structure for every 20 to 50 installed structures. However, the actual number of structures to be surveyed may be larger and should reflect what in each case is deemed necessary in order to obtain the required representativeness of the survey. In this respect all potential sources for variation from structure to structure should be considered. One issue to consider in this respect is the relative positioning and distances between the structures in a wind farm together with differences in environmental conditions. Another issue is the different times of installation of the structures. In large wind farms the installation period may span more than one year.

9.7.1.3 The few representative structures that are selected for inspection according to a defined risk-based plan shall be subjected to a condition-based monitoring, where critical components are inspected and checked for possible deficiencies. In the case that deficient structures or structural components are detected in one or more of the inspected representative structures, inspections of more structures in the wind farm shall be carried out.

9.7.1.4 Regarding inspection for fatigue cracks in steel structures, reference is made to DNVGL-RP-C210.

9.7.1.5 For grouted connections it is important to inspect the grout seal, which ensures confinement, for cracks and loss of grout at the top and bottom of the connections. This is particularly important for connections where bending moments are transferred through the grout. For conical-shaped connections it is important to check that the amount of settlements is as expected. It is usually sufficient to inspect a limited number of structures as long as it is found that the behaviour is as inspected for all structural parts of those inspected. Otherwise, the inspection should be extended to a larger number of structures.

9.8 Deviations

9.8.1 General

9.8.1.1 Deviations or non-conformances are findings made during an inspection that require special follow-up. Deviations may be assigned one of three different levels of concern according to their criticality:

- 1) Those impairing the overall safety, integrity and fitness of the installation or parts thereof and/or the persons on-board.
- 2) Those which are found to present a hazard for the persons on-board due to deterioration and/or damage or both; and those where documents for completing a matter are missing.
- 3) Those which are found to represent start of deterioration and those which are found to consist of minor defects.

The deviations shall be handled and reported accordingly.

APPENDIX A LOCAL JOINT FLEXIBILITIES FOR TUBULAR JOINTS

A.1 Calculation of local joint flexibilities

A.1.1 General

A.1.1.1 Calculation of local joint flexibilities (LJFs) for simple planar tubular joints can be carried out by application of available closed form solutions. Buitrago’s parametric expressions for LJFs should be used. These expressions give local joint flexibilities of brace ends for axial loading, for in-plane bending and for out-of-plane bending. There are expressions for single-brace joints (Y joints), for cross joints (X joints), and for gapped K joints and overlapped K joints. The expressions are given in terms of a number of geometric parameters whose definitions are given in Figure A-1. LJFs influence the global static and dynamic structural response.

A.1.1.2 In addition to direct flexibility terms between loading and deformation of a particular brace end, there are cross terms between loading of one brace end and deformation of another brace end in joints where more than one brace join in with the chord beam. Figure A-1 provides information of degrees of freedom for which cross terms of local joint flexibility exist between different brace ends.

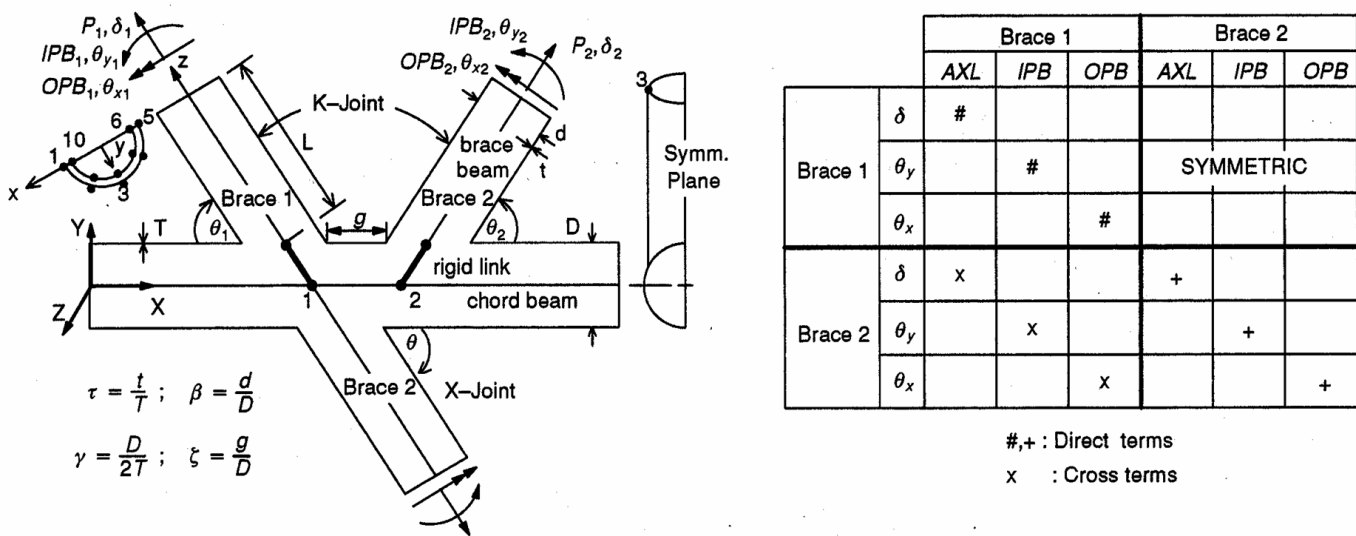


Figure A-1 General joint geometry, loads, and degrees of freedom

A.1.1.3 The local joint flexibility LJF for a considered degree of freedom of a brace end is defined as the net local deformation of the brace-chord intersection (footprint) in the brace local coordinates due to a unit load applied to the brace end.

A.1.1.4 The local joint flexibilities are expressed in terms of non-dimensional local joint flexibilities, f , which are also known as non-dimensional influence factors, as follows:

$$LJF_{axial} = \frac{f_{axial}}{ED}$$

$$LJF_{IPB} = \frac{f_{IPB}}{ED^3}$$

$$LJF_{OPB} = \frac{f_{OPB}}{ED^3}$$

in which E denotes Young's modulus, D is the outer chord diameter, IPB denotes in-plane bending, and OPB denotes out-of-plane bending. Expressions for f_{axial} , f_{IPB} and f_{OPB} are given in the following for various types of joints.

A.1.1.5 Implementation of LJFs in conventional frame analysis models requires springs, whose spring stiffnesses are equal to the inverse of the local joint flexibilities, to be included between the brace end and the corresponding point on the chord surface. Alternatively, a short flexible beam element can be included between the brace end and the chord at the chord surface.

A.1.1.6 LJFs are given separately for different joint types. However, note that for multi-brace joints, such as X and K joints, the LJFs are dependent on the load pattern. This implies that for a given load case, the joint should be classified by the loads or the load pattern, rather than by its actual geometry. This further implies that a multi-brace joint may be classified as a different joint type than the one which is given by its geometry, or it may be classified as a combination of joint types. In the former case, its LJFs shall be calculated according to the formulae given for the joint type to which the joint has become classified. In the latter case, its LJFs shall be calculated as:

$$\overline{LJF} = |\lambda_y LJF_y + \lambda_x LJF_x + \lambda_K LJF_K|$$

in which the λ values are the fractions corresponding to the joint type designated by the subscript when the joint is classified by loads.

A.1.1.7 It is important to include LJFs not only in joints which are being analysed, but also in joints which influence the force distribution at the joints which are being analysed.

A.1.1.8 The expressions for LJFs are developed for planar joints. For fatigue assessments in a traditionally braced jacket structure, the expressions can be applied to multi-planar joints as well, as long as these joints are un-stiffened and non-overlapping.

A.1.1.9 According to the above, the following steps should thus be included in a global analysis of a wind turbine support structure, based on a conventional frame analysis model of beam elements:

- 1) Classification of joints (T/Y/X/XT joints) by load pattern, i.e. not by geometry.
- 2) Implementation of local joint flexibility in all joints according to classification and parametric expressions by Buitrago.
- 3) Calculation of sectional forces at the surface footprint of the brace-to-chord connection.

A.1.1.10 The parametric expressions for calculation of LJFs for tubular joints are given in the following.

Table A-1 Non-dimensional influence factor expressions for local joint flexibility of single-brace joints

SINGLE-BRACE JOINTS

$$f_{axl} = 5.69 \tau^{-0.111} \exp(-2.251\beta) \gamma^{1.898} \sin^{1.769} \theta$$

$$f_{ipb} = 1.39 \tau^{-0.238} \beta^{-2.245} \gamma^{1.898} \sin^{1.240} \theta$$

$$f_{opb} = 55 \tau^{-0.220} \exp(-4.076\beta) \gamma^{2.417} \sin^{1.883} \theta$$

Table A-2 Non-dimensional influence factor expressions for local joint flexibility of X joints

CROSS JOINTS

$$f_{axl_1}^{\delta_1} = 8.94 \tau^{-0.198} \exp(-2.759 \beta) \gamma^{1.791} \sin^{1.700} \theta$$

$$f_{ipb_1}^{\theta y_1} = 67.60 \tau^{-0.063} \exp(-4.056 \beta) \gamma^{1.892} \sin^{1.255} \theta$$

$$f_{opb_1}^{\theta x_1} = 73.95 \tau^{-0.300} \exp(-4.478 \beta) \gamma^{2.367} \sin^{1.926} \theta$$

$$f_{axl_1}^{\delta_2} = \tau^{-0.1} (-353 + 1197 \beta - 1108 \beta \sin \theta - 40 \beta \gamma + 50 \gamma \sin \theta)$$

$$f_{ipb_1}^{\theta y_2} = \tau^{-0.1} (26 - 75 \beta^2 - 8.5 \beta^2 \sin \theta + 85 \beta^2 \gamma - 7.4 \gamma \sin \theta)$$

$$f_{opb_1}^{\theta x_2} = \tau^{-0.1} (2249 - 5879 \beta + 5515 \beta \sin \theta + 221 \beta \gamma - 358 \gamma \sin \theta)$$

Table A-3 Non-dimensional influence factor expressions for local joint flexibility of K joints

a) GAPPED JOINTS

$$f_{axl_1}^{\delta_1} = 5.90 \tau^{-0.114} \exp(-2.163\beta) \gamma^{1.869} \zeta^{0.009} \sin^{1.869} \theta_1 \sin^{-0.089} \theta_2$$

$$f_{ipb_1}^{\theta y_1} = 52.2 \tau^{-0.119} \exp(-3.835\beta) \gamma^{1.934} \zeta^{0.011} \sin^{1.417} \theta_1 \sin^{-0.108} \theta_2$$

$$f_{opb_1}^{\theta x_1} = 49.7 \tau^{-0.251} \exp(-4.165\beta) \gamma^{2.449} \zeta^{0.004} \sin^{1.865} \theta_1 \sin^{0.054} \theta_2$$

$$f_{axl_1}^{\delta_2} = 3.93 \tau^{-0.113} \exp(-2.198\beta) \gamma^{1.847} \zeta^{-0.056} \sin^{0.837} \theta_1 \sin^{0.784} \theta_2$$

$$f_{ipb_1}^{\theta y_2} = f_{ipb_1}^{\theta y_1} - 1.83 \tau^{-0.212} \beta^{-2.102} \gamma^{1.872} \zeta^{0.020} \sin^{1.249} \theta_1 \sin^{0.060} \theta_2$$

$$f_{opb_1}^{\theta x_2} = 4.37 \tau^{-0.295} \exp(-3.814\beta) \gamma^{2.875} \zeta^{-0.149} \sin^{0.885} \theta_1 \sin^{1.109} \theta_2$$

δ and θy and θx = Axial Deflection and IPB and OPB Rotations
Subscripts 1 and 2 = Brace 1 and Brace 2

b) OVERLAPPED JOINTS

$$f_{axl_1}^{\delta_1} = 3.91 \exp(-2.265\beta) \gamma^{2.010} \zeta^{-0.009} \sin^{1.811} \theta_1 \sin^{-0.029} \theta_2$$

$$f_{ipb_1}^{\theta y_1} = 1.86 \beta^{-2.093} \gamma^{1.766} \zeta^{-0.029} \sin^{0.711} \theta_1 \sin^{0.036} \theta_2$$

$$f_{opb_1}^{\theta x_1} = 54.2 \exp(-3.959\beta) \gamma^{2.403} \zeta^{0.001} \sin^{1.856} \theta_1 \sin^{-0.009} \theta_2$$

$$f_{axl_1}^{\delta_2} = 0.48 \beta^{-1.269} \gamma^{2.032} \zeta^{0.072} \sin^{0.949} \theta_1 \sin^{0.954} \theta_2$$

$$f_{ipb_1}^{\theta y_2} = 0.75 \beta^{-3.000} \gamma^{2.063} \zeta^{1.079} \sin^{0.533} \theta_1 \sin^{0.586} \theta_2$$

$$f_{opb_1}^{\theta x_2} = 1.16 \beta^{-2.068} \gamma^{2.550} \zeta^{0.117} \sin^{1.090} \theta_1 \sin^{1.089} \theta_2$$

ζ = Absolute value of g/D
 $f_{axl} = L J F_{axl} * E D$; $f_{ipb} = L J F_{ipb} * E D^3$; $f_{opb} = L J F_{opb} * E D^3$

APPENDIX B CROSS-SECTION TYPES

B.1 Cross-section types

B.1.1 General

B.1.1.1 Cross-sections of beams are divided into different types dependent of their ability to develop plastic hinges as given in Table B-1.

Table B-1 Cross-sectional types

Type	Description
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

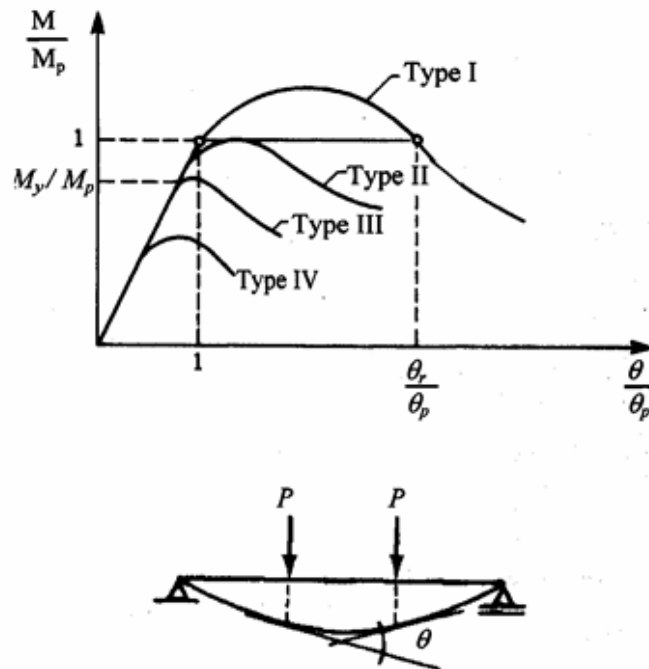


Figure B-1 Relation between moment M and plastic moment resistance M_p , and rotation θ for cross-sectional types. M_y is elastic moment resistance

B.1.1.2 The categorisation of cross-sections depends on the proportions of each of its compression elements, see Table B-3.

B.1.1.3 Compression elements include every element of a cross-section which is either totally or partially in compression, due to axial force or bending moment, under the load combination considered.

B.1.1.4 The various compression elements in a cross-section such as web or flange, can be in different classes.

B.1.1.5 The selection of cross-sectional type is normally quoted by the highest or less favourable type of its compression elements.

B.1.2 Cross-section requirements for plastic analysis

B.1.2.1 At plastic hinge locations, the cross-section of the member which contains the plastic hinge shall have an axis of symmetry in the plane of loading.

B.1.2.2 At plastic hinge locations, the cross-section of the member which contains the plastic hinge shall have a rotation capacity not less than the required rotation at that plastic hinge location.

B.1.3 Cross-section requirements when elastic global analysis is used

B.1.3.1 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.

B.1.3.2 When all the compression elements of a cross-section are type III, its resistance may be based on an elastic distribution of stresses across the cross-section, limited to the yield strength at the extreme fibres.

Table B-2 Coefficient related to relative strain

<i>NV Steel grade</i> ¹⁾	ϵ ²⁾
NV-NS	1
NV-27	0.94
NV-32	0.86
NV-36	0.81
NV-40	0.78
NV-420	0.75
NV-460	0.72

1) The table is not valid for steel with improved weldability. See DNVGL-OS-C101 Ch.2 Sec.3, Table 3 footnote 1.

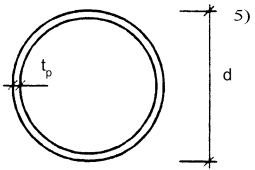
2)

$$\epsilon = \sqrt{\frac{235}{f_y}} \text{ where } f_y \text{ is yield strength in N/mm}^2$$

Table B-3 Maximum width-to-thickness ratios for compression elements

Cross-section part	Type I	Type II	Type III
<p>d</p> <p>t_w</p> <p>$d = h - 3 t_w$³⁾</p>	<p>¹⁾</p> <p>$d / t_w \leq 33 \epsilon$²⁾</p>	<p>$d / t_w \leq 38 \epsilon$</p>	<p>$d / t_w \leq 42 \epsilon$</p>
	<p>$d / t_w \leq 72 \epsilon$</p>	<p>$d / t_w \leq 83 \epsilon$</p>	<p>$d / t_w \leq 124 \epsilon$</p>
	<p>when $\alpha > 0.5$:</p> $d / t_w \leq \frac{396 \epsilon}{13 \alpha - 1}$ <p>when $\alpha \leq 0.5$:</p> $d / t_w \leq \frac{36 \epsilon}{\alpha}$	<p>when $\alpha > 0.5$:</p> $d / t_w \leq \frac{456 \epsilon}{13 \alpha - 1}$ <p>when $\alpha \leq 0.5$:</p> $d / t_w \leq \frac{41.5 \epsilon}{\alpha}$	<p>when $\psi > -1$:</p> $d / t_w \leq \frac{126 \epsilon}{2 + \psi}$ <p>when $\psi \leq -1$:</p> $d / t_w \leq 62 \epsilon (1 - \psi) \sqrt{ \psi }$

Cross-section part	Type I	Type II	Type III
	<p><i>Rolled:</i> $c/t_f \leq 10 \varepsilon$</p> <p><i>Welded:</i> $c/t_f \leq 9 \varepsilon$</p>	<p><i>Rolled:</i> $c/t_f \leq 11 \varepsilon$</p> <p><i>Welded:</i> $c/t_f \leq 10 \varepsilon$</p>	<p><i>Rolled:</i> $c/t_f \leq 15 \varepsilon$</p> <p><i>Welded:</i> $c/t_f \leq 14 \varepsilon$</p>
	<p>Tip in compression</p> <p><i>Rolled:</i> $c/t_f \leq 10 \varepsilon / \alpha$</p> <p><i>Welded:</i> $c/t_f \leq 9 \varepsilon / \alpha$</p>	<p>Tip in compression</p> <p><i>Rolled:</i> $c/t_f \leq 11 \varepsilon / \alpha$</p> <p><i>Welded:</i> $c/t_f \leq 10 \varepsilon / \alpha$</p>	<p>Tip in compression</p> <p><i>Rolled:</i> $c/t_f \leq 23 \varepsilon \sqrt{C}^{(4)}$</p> <p><i>Welded:</i> $c/t_f \leq 21 \varepsilon \sqrt{C}$</p>
	<p>Tip in tension</p> <p><i>Rolled:</i> $c/t_f \leq \frac{10 \varepsilon}{\alpha \sqrt{\alpha}}$</p> <p><i>Welded:</i> $c/t_f \leq \frac{9 \varepsilon}{\alpha \sqrt{\alpha}}$</p>	<p>Tip in tension</p> <p><i>Rolled:</i> $c/t_f \leq \frac{11 \varepsilon}{\alpha \sqrt{\alpha}}$</p> <p><i>Welded:</i> $c/t_f \leq \frac{10 \varepsilon}{\alpha \sqrt{\alpha}}$</p>	<p>Tip in tension</p> <p><i>Rolled:</i> $c/t_f \leq 23 \varepsilon \sqrt{C}$</p> <p><i>Welded:</i> $c/t_f \leq 21 \varepsilon \sqrt{C}$</p>

Cross-section part	Type I	Type II	Type III
	$d / t_p \leq 50 \varepsilon^2$	$d / t_p \leq 70 \varepsilon^2$	$d / t_p \leq 90 \varepsilon^2$
<ol style="list-style-type: none"> 1) Compression negative 2) ε is defined in Table B-2 3) Valid for rectangular hollow sections (RHS) where h is the height of the profile 4) C is the buckling coefficient. See EN 1993-1-1 Table 5.3.3 (denoted k_σ) 5) Valid for axial and bending, not external pressure. 			

APPENDIX C DESIGN OF GROUTED CONNECTIONS

C.1 Analytical verification methods

C.1.1 ULS for tubular and conical grouted connections without shear keys

C.1.1.1 The bending moment capacity consists of a component formed by radial contact pressures, a component formed by horizontal shear resistance between steel and grout, and a component formed by vertical shear resistance between steel and grout. The two shear resistance components consist of frictional resistance which are formed by a steel-to-grout friction coefficient in conjunction with the radial contact pressures in the zones where these contact pressures are transferred. In addition, there will be a contribution to the moment capacity from vertical friction resistance formed by surface irregularities such as fabrication tolerances in the grouted connection; however, it is recommended not to count on this contribution to the moment capacity in design, although it is important to consider it when planning laboratory tests on grouted connections and assessing laboratory test data from such tests.

C.1.1.2 Unless data indicate otherwise, the maximum nominal contact pressure $p_{nom,M}$ at the top and at the bottom of the grouted connection, caused by an applied bending moment M , may be found from the following expression:

$$p_{nom,M} = \frac{3\pi M}{R_p L_g^2 (\pi + 3\mu) + 3\pi \mu R_p^2 L_g}$$

where μ is the friction coefficient. $L_g = L - 2 \cdot t_g$ is the effective length of the grouted section, L is the full length of the grouted section from the grout packers to the outlet holes (or the top of the monopile), and t_g is the grout thickness. In this expression, the dependency on a horizontal shear force on the grouted connection is assumed to be insignificant. This assumption is valid for grouted connections for monopiles. For tubular (cylindrical) connections, R_p shall be taken as the outer radius of the innermost tube in the connection, For conical connections, R_p shall be taken as the average of the outer radius of the innermost cone in the connection as calculated over the area of this cone. Wherever the pressure from a shear force is not insignificant, the effects of this shear force shall be included in the design analysis. In such cases, this pressure $p_{nom,Q}$ may be estimated as:

$$p_{nom,Q} = \frac{Q}{2R_p * L_g}$$

where Q denotes the shear force.

The maximum nominal contact pressure due to bending moment M and shear force Q is defined as:

$$p_{nom} = p_{nom,M} + p_{nom,Q}$$

C.1.1.3 When the expression for the maximum nominal contact pressure in [C.1.1.2] is used to estimate the design value of the maximum nominal contact pressure for use in design, a characteristic value of the friction coefficient μ shall be used in conjunction with the design value of the applied bending moment M . The characteristic value of the friction coefficient μ is defined as the 5% quantile in the distribution of the coefficient and can be set to 0.7, unless data indicate otherwise. A higher value than 0.7 can be used if properly documented based on measurements from similar design and fabrication.

C.1.1.4 Local stress increase due to the discontinuities at the ends of the grouted section shall be accounted for in design. The local design contact pressure can be obtained from the design value of the maximum nominal contact pressure by multiplication by an appropriate stress concentration factor:

$$p_{local,d} = SCF * p_{nom,d}$$

with:

$$p_{nom,d} = p_{nom,Md} + p_{nom,Qd}$$

where an analytical result for the stress concentration factor SCF reads:

$$SCF = 1 + 0.025 * \left(\frac{R}{t}\right)^{3/2} ; 2250mm \leq R \leq 3250mm \text{ and } 50mm \leq t \leq 100mm.$$

This analytical result can be used unless other stress concentration factors can be documented. For grout at the end of the innermost tube or cone in the connection (the pile), the radius R and the wall thickness t shall be taken as the radius R_p and the wall thickness t_p of the pile. For grout at the end of the outermost tube or cone in the connection (the sleeve or the transition piece), the radius R and the wall thickness t shall be taken as the radius R_s/R_{TP} and the wall thickness t_s/t_{TP} of the sleeve or transition piece, as applicable. Alternatively, the local design contact pressure can be obtained from a local linear finite element analysis. Such local linear finite element analysis needs to be calibrated, see [6.5.1.6]. In this analysis, the grout material shall be modelled as linear, whereas contact elements can be nonlinear.

C.1.1.5 The design tensile stress in the grout shall be taken as:

$$\sigma_d = 0.25 * p_{local,d} \left(\sqrt{1 + 4\mu_{local}^2} - 1 \right)$$

where μ_{local} is a local friction coefficient representative for the contact areas at the top and the bottom of the grouted connection, and $p_{local,d}$ is the design value of the local contact pressure p_{local} . A characteristic value of p_{local} shall be used, which in this case is defined as the mean value.

C.1.1.6 The design criterion against local fracture in tension is:

$$\sigma_d \leq \frac{f_{tn}}{\gamma_m}$$

and the criterion shall be satisfied at either end of the grouted connection.

C.1.1.7 Torque capacities of tubular grouted connections are mainly formed by resistance against sliding between grout and steel and by surface irregularities in the connections. When torque is transferred from the transition piece to the pile, this sliding resistance may be counted on in design if combined with regular inspections of rotation in operation. The torque capacity that can be counted on may then be taken as the capacity that results from the vertical load in combination with friction.

C.1.2 Special provisions for conical grouted connections in monopiles without shear keys

C.1.2.1 The introduction of a cone angle in the grouted connection represents an introduction of well-defined minimum fabrication tolerances, such that agreement with assumptions made in design can be counted on. By constructing the grouted connection from two well-defined steel cones, permanent vertical displacements due to axial loading can be limited, and an axial capacity of the connection can be counted on in design, in contrast to what holds for tubular grouted connections without shear keys. Small cone angles are assumed such that the grouted connection can be assumed to transfer bending moments by compression in the grout in a manner similar to the way in which tubular grouted connections function. Use of a small cone angle in the range 1° to 3° is recommended.

C.1.2.2 For cone angles less than 4°, the moment capacity of a conical grouted connection can be reckoned as similar to that of a tubular grouted connection. The local pressures in the grout at the ends of a conical grouted connection will not be significantly different from those at the ends of a tubular grouted connection; however, there might be some additional stress owing to permanent vertical displacements caused by axial loading. Such additional stresses may be avoided by using tubular terminations of the two steel cones at either end of the grouted connection instead of having a conical shape of the grouted connection over the full height of the grouted section.

C.1.2.3 The permanent contact pressure between the steel surfaces and the grout in a conical grouted connection, owing to the weight of the turbine and the tower above the connection, results in a design torque capacity which may be expressed as:

$$M_{t,d} = \frac{2}{\gamma_m} p_{nom,d} \mu L_g \left(R_{pt}^2 + R_{pt} L_g \sin \alpha + \frac{L_g^2}{3} \sin^2 \alpha \right)$$

where p_{nom} is the nominal contact pressure between steel and grout due to the weight of the turbine and tower, R_{pt} is the radius of the connection on top of the pile (the inner cone), $L_g = L - 2 \cdot t_g$ is the effective length of the grouted section, L is the full length of the grouted section, measured along the cone surface from the grout packers to the outlet holes at the top of the inner cone, t_g is the grout thickness, and α is the cone angle, i.e. the angle with the vertical axis of the cone. The friction coefficient μ should be represented by its mean value. For design of a conical grouted connection against torque, the material factor γ_m shall be taken as 1.0, as there is some reserve capacity due to surface irregularities.

C.1.2.4 For a conical grouted connection, it is a main purpose of the grout to set up a pressure between the sleeve and the pile if the sleeve tends to slide downwards relative to the pile. The design should therefore allow for some vertical settlement δ_v . For a vertical settlement δ_v there will be a horizontal displacement δ :

$$\delta = \delta_v \cdot \tan \alpha$$

where α is the cone angle measured from the vertical plane. The pressure between steel and grout can be calculated as:

$$p_{nom} = \frac{E \cdot \delta}{F \cdot R_p}$$

where the flexibility F is:

$$F = \left[\frac{R_p}{t_p} + \frac{t_g E}{E_g R_p} + \frac{R_{TP}}{t_{TP}} \right]$$

and where R_p is outer radius of pile and R_{TP} is outer radius of transition piece, both averaged over the height of the grouted connection. Further, t_p is wall thickness of pile, t_{TP} is wall thickness of transition piece, t_g is thickness of grout, E is Young's modulus for steel and E_g is Young's modulus for grout.

Equilibrium for a vertical weight P_g acting on the grouted connection implies:

$$P = \frac{P_g}{\mu \cos \alpha + \sin \alpha}$$

where P is the total reaction force to be transferred through the grout. The friction coefficient μ should be represented by its characteristic value. The pressure p_{nom} acting on the outside grout area A_{cone} is:

$$p_{nom} = \frac{P}{A_{cone}}$$

The outside grout area A_{cone} is expressed as:

$$A_{cone} = 2\pi L_g R_{pt} + \pi L_g^2 \sin \alpha$$

When the pressure p_{nom} is known, the resulting displacement d and the resulting settlement δ_v may be calculated from the above expressions.

C.1.2.5 The available friction capacity cannot be utilized fully to carry the applied torque according to [C.1.2.3] and at the same time also be utilized fully to carry the applied dead load according to [C.1.2.4]. At high utilization, a vector addition of the horizontal torque effect and the vertical dead load effect should be carried out prior to the capacity check. In case of a large torque in combination with insufficient capacity, application of vertical shear keys at the midlevel of the grouted connection in order to carry the torque should be considered.

C.1.3 ULS for tubular grouted connections with shear keys – monopile

C.1.3.1 The following assumptions are prerequisites for the analytical design procedure.

- All shear keys are assumed to have the same distance s and height h .
- The arrangement of the shear keys on the TP and the pile has to adhere to Figure 6-2 in Sec.6.
- The number of shear keys on the TP and the number of shear keys of the pile differ by one.
- The grout layer should be homogeneously filled with grout material.

C.1.3.2 Unless data indicate otherwise, the maximum nominal radial contact pressure $p_{nom,d}$ at the top and at the bottom of the grouted connection, caused by an applied bending moment M_d , shall be derived from the following expression:

$$p_{nom,d} = \frac{3\pi M_d E L_g}{E L_g * \{R_p L_g^2 (\pi + 3\mu) + 3\pi \mu R_g^2 L_g\} + 18\pi^2 k_{eff} R_p^3 \left\{ \frac{R_p^2}{t_p} + \frac{R_{TP}^2}{t_{TP}} \right\}}$$

where:

k_{eff} = effective spring stiffness for the shear keys

μ = characteristic friction coefficient, equal to 0.7

R_p = outer radius of the pile

R_{TP} = outer radius of transition piece

t_p = wall thickness of pile

t_{TP} = wall thickness of transition piece

L_g = $L - 2 \cdot t_g$ = effective length of grouted section

L = full length of grouted section from the grout packers to the top of the pile

t_g = nominal grout thickness

C.1.3.3 The effective spring stiffness per unit length around the circumference of the grouted connection for n shear keys is expressed as:

$$k_{eff} = \frac{2t_{TP}s_{eff}^2 n E \Psi}{4\sqrt[4]{3(1-\nu^2)} t_g^2 \left[\left(\frac{R_p}{t_p} \right)^{3/2} + \left(\frac{R_{TP}}{t_{TP}} \right)^{3/2} \right] t_{TP} + n s_{eff}^2 L_g}$$

where:

s_{eff} = effective vertical distance between shear keys = $s - w$

s = vertical centre-to-centre distance between two consecutive shear keys

w = width of shear key

E = Young's modulus for steel

ν = Poisson's ratio for steel

n = number of effective shear keys (the actual number of shear keys on each side of the grouted connection is $n+1$)

Ψ = design coefficient

= 1.0 for calculation of load action on shear keys

= 0.5 for calculation of maximum nominal radial contact pressure.

C.1.3.4 The action force per unit length along the circumference, owing to bending moment and vertical force and transferred to the shear keys, shall be taken as:

$$F_{V Shk,d} = \frac{6p_{nom,d} k_{eff} R_p}{E L_g} \left(\frac{R_p^2}{t_p} + \frac{R_{TP}^2}{t_{TP}} \right) + \frac{P_d}{2\pi R_p}$$

where:

P_d = self-weight of structure above the pile, including full weight of the transition piece.

C.1.3.5 The average action force per unit length along the circumference on one shear key shall be taken as:

$$F_{V1\ Shk,d} = \frac{F_{V\ Shk,d}}{n}$$

C.1.3.6 The characteristic interface shear capacity in the grouted connection with shear keys shall be taken as:

$$f_{bk} = \left[\frac{800}{D_p} + 140 \left(\frac{h}{s} \right)^{0.8} \right] k^{0.6} f_{ck}^{0.3}$$

where:

h = height of shear key measured radially from the grout-steel interface

D_p = pile diameter in units of mm

k = radial stiffness parameter defined as:

$$k = \left[(2R_p/t_p) + (2R_{TP}/t_{TP}) \right]^{-1} + (E_g/E) \left[(2R_{TP} - 2t_{TP})/t_g \right]^{-1}$$

E_g = Young's modulus for the grout

$f_{ck,cube75}$ = characteristic compressive strength of 75 mm cubes in units of MPa

s = vertical centre-to-centre distance between shear keys

However, the characteristic interface shear capacity shall not be taken larger than the limit set forth by grout matrix failure:

$$f_{bk} = \left[0.75 - 1.4 \left(\frac{h}{s} \right) \right] f_{ck}^{0.5}$$

Reference is made to [Figure 6-2](#), [Figure 6-4](#) and [Figure 6-5](#) in [Sec.6](#) for explanation of symbols.

C.1.3.7 The characteristic capacity per unit length of one shear key is:

$$F_{V1\ Shk\ cap} = f_{bk} \cdot s$$

The design capacity per unit length of one shear key is

$$F_{V1\ Shk\ cap,d} = \frac{F_{V1\ Shk\ cap}}{\gamma_m}$$

C.1.3.8 The following requirement for the geometry of the shear keys shall be fulfilled:

$$h \geq 5\ mm \quad 1.5 \leq \frac{w}{h} \leq 3.0 \quad \frac{h}{s} \leq 0.10$$

where s is the vertical centre-to-centre distance between the shear keys, h is the height of the shear keys and w is the width of the shear keys.

C.1.3.9 It is recommended that the grout-length-to-pile-diameter ratio is kept within the following range:

$$1.5 \leq \frac{L_g}{D_p} \leq 2.5$$

where:

$L_g = L - 2 \cdot t_g$ = effective length of grouted section

L = full length of grouted section from the grout packers to the top of the pile

t_g = nominal grout thickness

D_p = outer pile diameter.

C.1.3.10 The following requirement for the geometry of the monopile shall be fulfilled:

$$10 \leq \frac{R_p}{t_p} \leq 30$$

This requirement can be waived when a buckling analysis of the pile is performed in which the pile is loaded by the actions from the grout at the shear keys and/or contact pressure from ovalization of the pile due to global bending. This buckling analysis should include fabrication tolerances similar to the first buckling mode.

C.1.3.11 The following requirement for the geometry of the transition piece shall be fulfilled:

$$9 \leq \frac{R_{TP}}{t_{TP}} \leq 70$$

C.1.3.12 The following requirement for the vertical distance between shear keys shall be fulfilled:

$$s \geq \min \left\{ \begin{array}{l} 0.8 \sqrt{R_p t_p} \\ 0.8 \sqrt{R_{TP} t_{TP}} \end{array} \right\}$$

This requirement can be waived in favor of smaller distances for the purpose of fulfilling requirements for design of shear keys against the FLS. When such smaller distances are applied between shear keys, the verification of capacity of shear keys against the ULS shall still be carried out on the basis of the specified requirement for the vertical distance between the shear keys.

C.1.4 ULS for tubular grouted connections with shear keys – jacket

C.1.4.1 The following assumptions are prerequisites for the analytical design procedure.

- The following formulas can be applied for post-installed and pre-installed piles.
- All shear keys are assumed to have the same distance s and height h .
- The arrangement of the shear keys on the TP and the pile has to adhere to [Figure 6-2](#) in [Sec.6](#).
- The number of shear keys on the TP and the number of shear keys of the pile differ by one.

— The grout layer should be homogeneously filled with grout material.

C.1.4.2 The design load per unit length along the circumference of one shear key shall be taken as:

Post-installed piles:
$$F_{V1\ shk,d} = \frac{P_{a,d}}{2\pi R_p n}$$
 where R_p is the outer radius of the pile

Pre-installed piles:
$$F_{V1\ shk,d} = \frac{P_{a,d}}{2\pi R_{jL} n}$$
 where R_{jL} is the outer radius of the jacket leg

and $P_{a,d}$ is the design axial load acting on the connection, calculated from the applied characteristic loads and load factors specified for the ULS in [Sec.5](#), and n is the number of effective shear keys on either side of the grout in the connection, i.e. the number of shear keys in the region which is not significantly affected by the bending moment.

C.1.4.3 The interface shear capacity in the grouted connection with shear keys shall be taken as:

Post-installed pile:
$$f_{bk} = \left[\frac{800}{D_p} + 140 \left(\frac{h}{s} \right)^{0.8} \right] k^{0.6} f_{ck}^{0.3}$$

Pre-installed pile:
$$f_{bk} = \left[\frac{800}{D_{jL}} + 140 \left(\frac{h}{s} \right)^{0.8} \right] k^{0.6} f_{ck}^{0.3}$$

where

h = height of shear key measured radially from the grout-steel interface

D_p = pile diameter in units of mm

D_{jL} = jacket leg diameter in units of mm

$f_{ck,cube75}$ = characteristic compressive strength of 75 mm cubes in units of MPa

s = vertical centre-to-centre distance between shear keys

k = radial stiffness parameter defined as:

post-installed pile:
$$k = [(2R_p/t_p) + (2R_s/t_s)]^{-1} + (E_g/E)[(2R_s - 2t_s)/t_g]^{-1}$$

pre-installed pile:
$$k = [(2R_{jL}/t_{jL}) + (2R_p/t_p)]^{-1} + (E_g/E)[(2R_p - 2t_p)/t_g]^{-1}$$

where:

E_g = Young's modulus for the grout

R_s = outer radius of sleeve

R_p = outer radius of pile

R_{jL} = outer radius of jacket leg

t_s = wall thickness of sleeve

t_p = wall thickness of pile

t_{jL} = wall thickness of jacket leg

However, the interface shear capacity shall not be taken larger than the limit set forth by grout matrix failure:

$$f_{bk} = \left[0.75 - 1.4 \left(\frac{h}{s} \right) \right] f_{ck}^{0.5}$$

See Figure 6-4 and Figure 6-5 in Sec.6 for explanation of symbols.

C.1.4.4 The characteristic capacity per unit length of one shear key is:

$$F_{V1 Shk cap} = f_{bk} \cdot s$$

C.1.4.5 The design capacity per unit length of one shear key is:

$$F_{V1 Shk cap,d} = \frac{F_{V1 Shk cap}}{\gamma_m}$$

C.1.4.6 The following requirement for the vertical distance between shear keys shall be fulfilled:

$$\text{Post-installed pile: } s \geq \min \begin{cases} 0.8\sqrt{R_p t_p} \\ 0.8\sqrt{R_s t_s} \end{cases}$$

$$\text{Pre-installed pile: } s \geq \min \begin{cases} 0.8\sqrt{R_p t_p} \\ 0.8\sqrt{R_{JL} t_{JL}} \end{cases}$$

This requirement can be waived in favour of smaller distances for the purpose of fulfilling requirements for design of shear keys against the FLS. When such smaller distances are applied between shear keys, the verification of capacity of shear keys against the ULS shall be carried out on the basis of the given requirement for the vertical distance between the shear keys.

C.1.4.7 The following requirements for the geometry of the shear keys shall be fulfilled:

$$h \geq 5 \text{ mm} \quad 1.5 \leq \frac{w}{h} \leq 3.0 \quad \frac{h}{s} \leq 0.10$$

where s is the vertical centre-to-centre distance between the shear keys, h is the height of the shear keys and w is the width of the shear keys. It is recommended that $h/D_p \leq 0.012$ for the post-installed pile, resp. $h/D_{JL} \leq 0.012$ for the pre-installed pile is fulfilled, where D_p denotes the outer pile diameter and D_{JL} denotes the jacket leg diameter

C.1.4.8 It is recommended that the grout-length-to-pile-diameter ratio is kept within the following range:

$$\text{Post-installed pile: } 1 \leq \frac{L_g}{D_p} \leq 10$$

$$\text{Pre-installed pile: } 1 \leq \frac{L_g}{D_{JL}} \leq 10$$

where L_g denotes the effective length of the grouted section and D_p denotes the outer pile diameter. When $L_g/D_p < 2.5$, resp. $L_g/D_{JL} < 2.5$, then the design procedure in [C.1.3] for grouted connections in monopiles can be used as an alternative for documentation of grouted connections in jacket structures.

C.1.4.9 It is recommended that the grout dimensions meet the following limitations:

$$10 \leq \frac{D_g}{t_g} \leq 45$$

where D_g denotes the outer diameter of the grout and t_g denotes the nominal grout thickness.

C.1.4.10 The following requirement for the geometry of the pile shall be fulfilled:

Post-installed pile: $10 \leq \frac{R_p}{t_p} \leq 30$

Pre-installed pile: $10 \leq \frac{R_{JL}}{t_{JL}} \leq 30$

C.1.4.11 The following requirement for the geometry of the sleeve shall be fulfilled:

Post-installed pile: $15 \leq \frac{R_s}{t_s} \leq 70$

Pre-installed pile: $15 \leq \frac{R_p}{t_p} \leq 70$

C.1.4.12 For connections involving post-installed piles, the region which is significantly affected by the bending moment is the region of the connection from a level half an elastic length above the base of the connection and downwards.

The elastic length of the pile can be taken as

Post-installed pile: $l_e = \sqrt[4]{\frac{4EI_p}{k_{rD}}}$

where:

I_p = moment of inertia of the pile.

C.1.4.13 For connections involving pre-installed piles, the region which is significantly affected by the bending moment is the region of the connection from a level half an elastic length below the top of the connection and upwards.

The elastic length of the pile can be taken as:

Pre-installed pile: $l_e = \sqrt[4]{\frac{4EI_{JL}}{k_{rD}}}$

where

I_{JL} = moment of inertia of the jacket leg.

C.1.4.14 The supporting spring stiffness k_{rD} , defined as the radial spring stiffness times the pile diameter, may be expressed as:

Post-installed pile: $k_{rD} = \frac{4ER_p}{\frac{R_p^2}{t_p} + \frac{R_s^2}{t_s} + t_g m}$

Pre-installed pile:

$$k_{rD} = \frac{4ER_{JL}}{\frac{R_p^2}{t_p} + \frac{R_{JL}^2}{t_{JL}} + t_g m}$$

where:

E = Young's modulus for steel

R_p = radius to outer part of pile

R_s = radius to outer part of sleeve

R_{JL} = radius to outer part of jacket leg

t_p = thickness of pile

t_s = thickness of sleeve

t_{JL} = thickness of jacket leg

t_g = nominal thickness of grout

m = ratio of Young's modulus for steel and Young's modulus for grout material; m = 18 can be used if Young's modulus for grout material is not known.

C.1.4.15 Unless data indicate otherwise, the maximum nominal radial contact pressure p_{nom} at the grouted connection, caused by the design horizontal shear force Q_0 and the design bending moment M_0 at the bottom of the sleeve in case of post-installed piles, shall be derived from the following expression:

Post-installed pile:
$$\rho_{nom,d} = \frac{l_e^2 k_{rD}}{8EI_p R_p} * (M_d + Q_d * l_e)$$

Pre-installed pile:
$$\rho_{nom,d} = \frac{l_e^2 k_{rD}}{8EI_{JL} R_{JL}} * (M_d + Q_d * l_e)$$

where:

l_e = elastic length of pile as defined in [C.1.4.12]

k_{rD} = support spring stiffness as defined in [C.1.4.14]

E = Young's modulus for steel

I_p = moment of inertia of the pile I_{JL} = moment of inertia of the jacket leg

R_p = outer radius of the pile R_{JL} = outer radius of the jacket leg

C.1.4.16 Grouted connections of jacket structures (with preinstalled piles) often have large tolerances. When such large tolerances are in place, the moment due to eccentricity associated with the tolerances should be included in design in addition to the design bending moment M_0 .

C.1.5 ULS for tubular grouted connections with shear keys - torque

C.1.5.1 The design load per unit length of one vertical shear key is:

$$F_{H1\ Shk,d} = \frac{M_{T,d}}{R_p L_S n}$$

where $M_{T,d}$ is the applied design torque, R_p is the outer radius of the monopile, L_S is the length of one vertical shear key and n is the number of vertical shear keys on each side of the grout.

C.1.5.2 The characteristic interface shear capacity for torque in the grouted connection with vertical shear keys is:

$$f_{bk} = \left[\frac{800}{D_p} + 140 \left(\frac{h}{s} \right)^{0.8} \right] k^{0.6} f_{ck}^{0.3}$$

where the parameters are the same as those specified in [C.1.3.6], except for the parameter s which is here the horizontal distance between vertical shear keys, measured along the circumference of the monopile.

However, the characteristic interface shear capacity for torque in the grouted connection with vertical shear keys shall not be taken larger than the limit set forth by grout matrix failure:

$$f_{bk} = \left[0.75 - 1.4 \left(\frac{h}{s} \right) \right] f_{ck}^{0.5}$$

C.1.5.3 The characteristic capacity per unit length of one shear key is:

$$F_{H1\ Shk\ cap} = f_{bk} \cdot s$$

C.1.5.4 The design capacity per unit length of one shear key is:

$$F_{H1\ Shk\ cap,d} = \frac{F_{H1\ Shk\ cap}}{\gamma_m}$$

C.1.6 Fatigue limit states for tubular grouted connections with shear keys - general

C.1.6.1 Design against the FLS for the grouted connection is based on the expected long term load history. When the expected long term distribution of load amplitudes has been established, the fatigue design of the grouted connection shall be carried out according to the following procedure:

The S-N curve, which gives number of cycles N to failure at a specified relative load level y , is represented as:

$$\begin{array}{ll} \log N = 5.400 - 8y & \text{for } y \geq 0.30 \\ \log N = 7.286 - 14.286y & \text{for } 0.16 < y < 0.30 \\ \log N = 13.000 - 50y & \text{for } y \leq 0.16 \end{array}$$

where, for each load cycle, the relative load level is:

$$y = \frac{F_{V1\ Shk} \gamma_m}{F_{V1\ Shk\ cap}}$$

where $F_{V1\ Shk\ cap}$ is the characteristic shear key capacity as described in [C.1.3.8], $F_{V1\ Shk}$ is the shear key load of the load cycle in question, consisting of the static load and the load amplitude of the load cycle in question, and γ_m is the material factor.

C.1.6.2 Installation tolerances for the monopile and the transition piece will imply that the grout thickness will vary along the circumference of the grouted connection. When there are guides between the transition piece and the pile at the top of the monopile, the maximum tolerance on the grout thickness in the region of the shear keys becomes:

$$\Delta t_g = \frac{L}{2} \tan \varphi$$

where φ is the installation angle tolerance with respect to deviation between the axis of MP and TP. The effect of installation tolerance should be considered for design against the FLS. For this purpose, the action force on the shear keys per unit length along the circumference shall be taken as:

$$F_{VShk, mod} = \frac{6\rho_{nom} k_{eff, mod} R_p}{E L_g} \left(\frac{R_p^2}{t_p} + \frac{R_{TP}^2}{t_{TP}} \right) + \frac{P}{2\pi R_p}$$

where:

$$k_{eff, mod} = \frac{2t_{TP} s_{eff}^2 n E \Psi}{4\sqrt[4]{3(1-\nu^2)} t_{g, min}^2 \left[\left(\frac{R_p}{t_p} \right)^{3/2} + \left(\frac{R_{TP}}{t_{TP}} \right)^{3/2} \right] t_{TP} + n s_{eff}^2 L_g}$$

is a modified effective spring stiffness for the loading on the n shear keys, based on the minimum grout thickness $t_{g, min}$ calculated as:

$$t_{g, min} = t_g - \Delta t_g$$

The average action force per unit length along the circumference on one shear key shall be taken as:

$$F_{V1, Shk} = \frac{F_{V, Shk, mod}}{n}$$

Note that a local tolerance with deviation in position from nominal position equal to 5% of the distance between the shear keys is within the tolerance accounted for by the material factor used in design. A global mispositioning of all the shear keys on the transition piece relative to the pile can be accounted for in manner similar to the way in which mispositioning angle tolerance is accounted for above, if needed.

Guidance note:

The horizontal tolerance is zero at the top of the connection in case centrallizers are used at the top.

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

C.1.7 FLS for tubular grouted connections with shear keys in monopiles

C.1.7.1 The following requirement for vertical distance between shear keys shall be fulfilled:

$$s \geq \min \begin{cases} 0.4\sqrt{R_p t_p} \\ 0.4\sqrt{R_{TP} t_{TP}} \end{cases}$$

C.1.7.2 When the expression in [C.1.3.4], is applied for calculation of load amplitudes to be used in the fatigue evaluation, the amplitude values of F_{V1Shk} for the individual load cycles in [C.1.6.1], can be replaced by half the load range for those load cycles where the permanent load level due to deadweight is larger than the reversed upward load amplitude on the shear key from the environmental loading, see Figure C-1.

Guidance note:

Load cycles without load reversal and associated backwards sliding in the grouted connection are considered less critical with respect to fatigue than load cycles with load reversal causing backwards sliding. Load reversal can occur when the permanent axial load level due to deadweight is less than the reversed load amplitude on the shear key owing to environmental loading. For this situation F_{V1Shk} needs to be taken as the downward load amplitude for the calculation of number of cycles to failure in [C.1.6], see Figure C-1. When no load reversal can occur, it suffices to replace F_{V1Shk} in these calculations by half the load range. The S-N curve specified in [C.1.6.1] is developed for conditions with load reversal, and this replacement is partly meant to account for the effects of a more favourable S-N curve when no load reversal takes place.

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

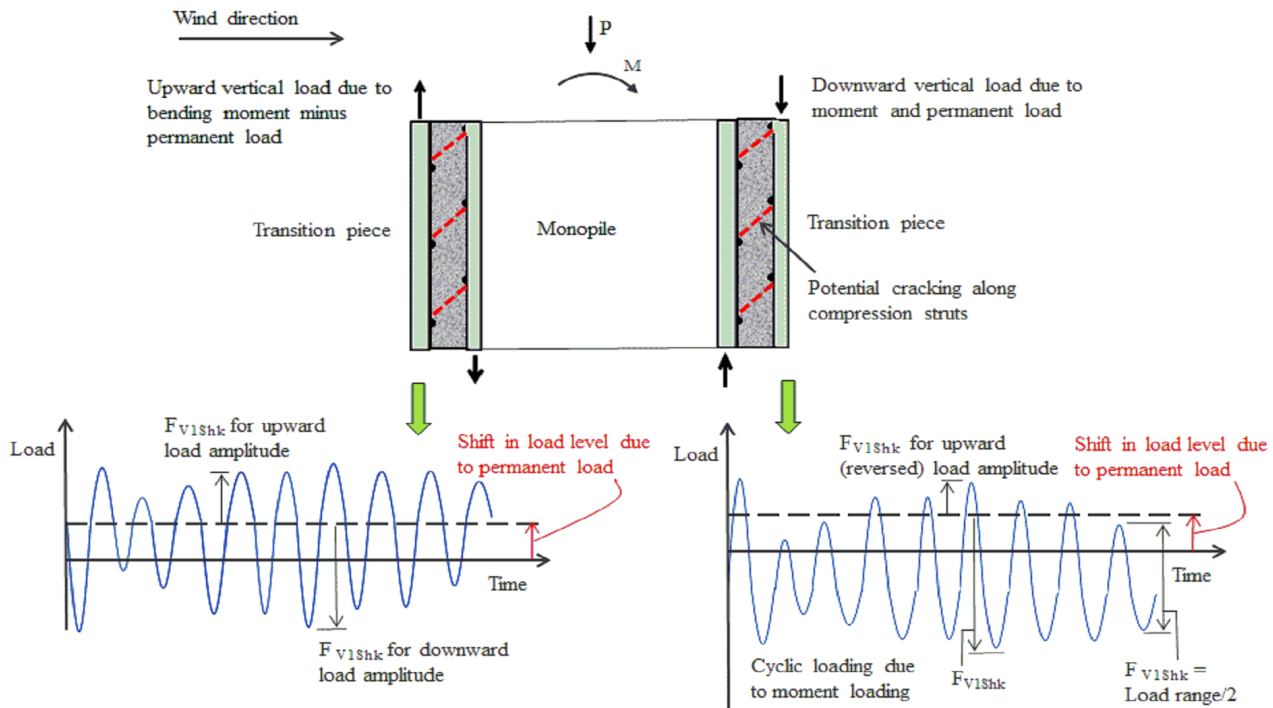


Figure C-1 Schematic loading on monopile with illustration of load amplitudes on shear keys to be used in fatigue analysis

C.1.8 FLS for tubular grouted connections with shear keys in jacket structures

C.1.8.1 The following requirement for vertical distance between shear keys in jacket structures with postinstalled piles shall be fulfilled:

$$s \geq \min \begin{cases} 0.4\sqrt{R_p t_p} \\ 0.4\sqrt{R_s t_s} \end{cases}$$

C.1.8.2 The following requirement for vertical distance between shear keys in jacket structures with preinstalled piles shall be fulfilled:

$$s \geq \min \begin{cases} 0.4\sqrt{R_p t_p} \\ 0.4\sqrt{R_{JL} t_{JL}} \end{cases}$$

C.1.8.3 When the long term history of load amplitudes to be used for fatigue design has been established according to the specifications in [6.6.1], the amplitude values of F_{V1Shk} for the individual load cycles in

[C.1.6.1], can be replaced by half the load range for those load cycles where the permanent load level due to deadweight is greater than the reversed load amplitude, see Figure C-2.

Guidance note:

Load cycles without load reversal and associated backwards sliding in the grouted connection are considered less critical with respect to fatigue than load cycles with load reversal causing backwards sliding. Load reversal can occur when the static axial load level is less than the reversed load amplitude owing to environmental loading. For this situation F_{V1Shk} needs to be taken as the downward load amplitude for the calculation of number of cycles to failure in [C.1.6.1], see Figure C-2. When no load reversal can occur, it suffices to replace F_{V1Shk} in these calculations by half the load range. The S-N curve specified in [C.1.6.1] is developed for conditions with load reversal, and this replacement is partly meant to account for the effects of a more favorable S-N curve when no load reversal takes place.

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

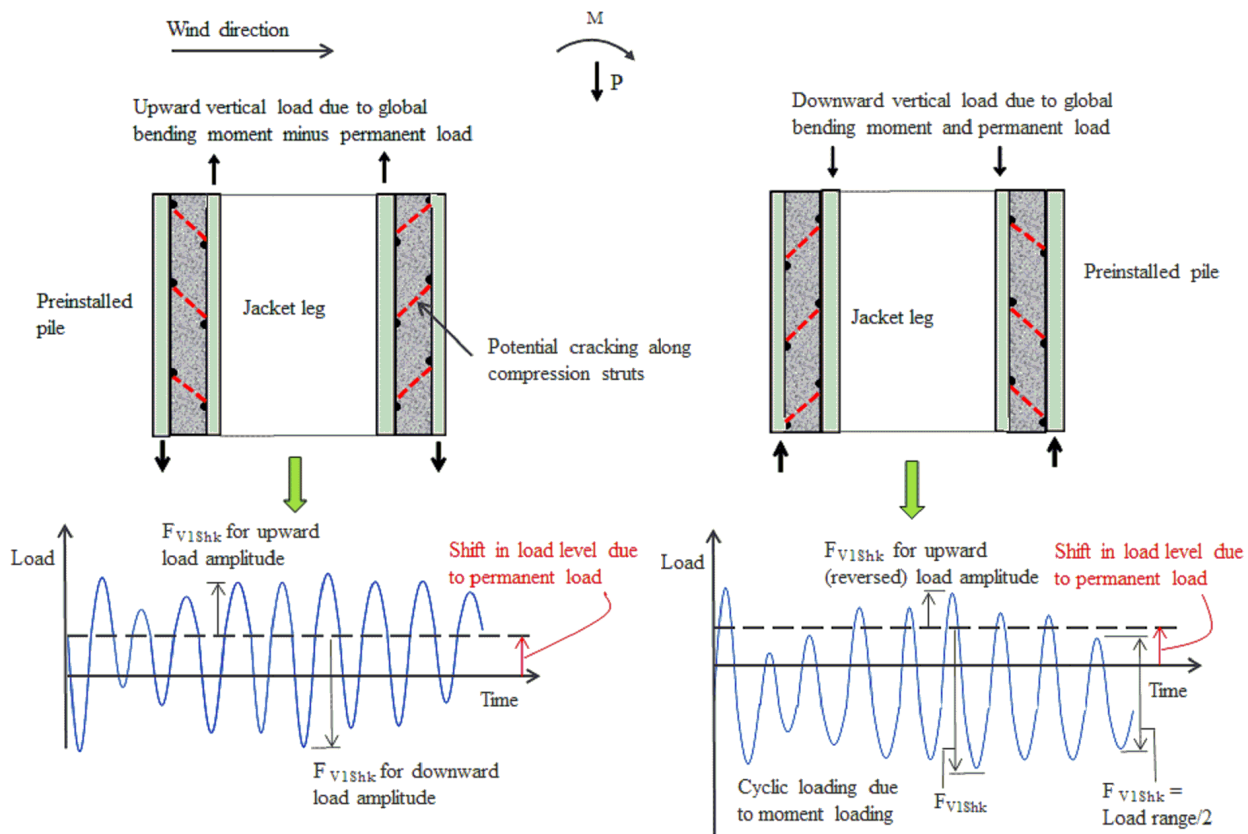


Figure C-2 Schematic loading on preinstalled piles with illustration of load amplitudes from jacket legs on shear keys to be used in fatigue analysis

C.2 Numerical verification methods

C.2.1 General

C.2.1.1 In general, it is recommended to first perform an analytical analysis according to [C.1] and only in case the requirements lead to a non-favourable design solution a numerical analysis by means of a finite element analysis (FEA) can be used for assessment of ultimate and fatigue limit capacity of grouted connections. For this case [C.2] is giving guidance on how to find a safe solution for the design of a grouted connection by means of FEA. Rules for application and recommendations are furthermore given in the DNVGL-RP-0419. Several parameters required for assessment of capacity of grouted connections by use of finite element analysis are encumbered with uncertainty. Uncertainties are associated with element types, element mesh in the region of the highest stresses, friction coefficient, characteristics of the grout material, material modelling, contact formulation, and convergence criterion.

Guidance note:

In case of applying a FEA it should be considered that calculation of fatigue capacity of a grouted connection is still matter of research. For the designer it should be clear that uncertainties exist for the time being which should be covered by assumptions made leading to a safe design. The application of FEA requires a large amount of experience and knowledge of software and grout material.

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

C.2.2 Ultimate limit states for tubular grouted connections with shear keys

C.2.2.1 The grout material shall be verified against ultimate loads by evaluating a state of equilibrium using a validated material law.

C.2.2.2 The grout material shall be verified against extreme loads using the maximum compression stress $\sigma_{3,FEA}$ (minimum principle) from the finite element calculation (FEA).

Guidance note:

Detailed information on how to apply a FEA is given in DNVGL-RP-0419.

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

C.2.2.3 The design criterion for the grouted connection with shear keys is

$$|\sigma_{3,FEA}| \leq f_{cd}$$

C.2.3 Serviceability limit states for tubular grouted connections with shear keys

C.2.3.1 It shall be checked that application of SLS load is not leading to significant cracking in order to avoid load redistribution.

C.2.4 Fatigue limit states for tubular grouted connections with shear keys

C.2.4.1 The verification shall be performed by means of detailed Markov matrices.

C.2.4.2 The FLS-verification for the case without stress redistribution and degradation of grout stiffness can be done by damage accumulation. A linear damage accumulation according to Palmgren-Miner is assumed

for the grout material in this verification. Basis for FLS loading used for this verification is a Markov matrix including ranges of loads and corresponding mean values. Taking a transfer function between loads and stresses the Markov matrix can be transformed in an equivalent matrix of stresses for every location. Figure C-3 shows an example of the maximum compressive stress in the grout caused by load spectra for three mean values.

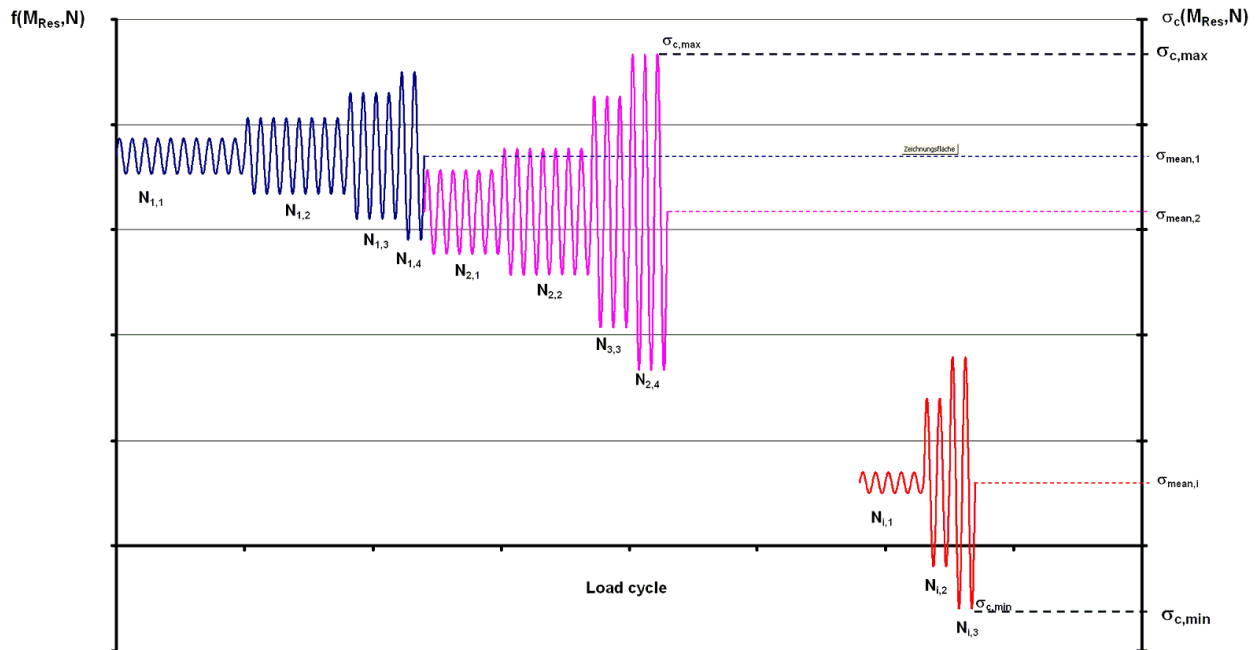


Figure C-3 Load spectra for the damage calculation according Palmgren/Miner

Guidance note:

It is required to determine a transfer function which reflects the relation between external (global) load ranges and stresses in steel and grout. The transfer function can be used to determine the compressive stress in the grout for each load level of the Markov matrix and perform a complete damage accumulation. The transfer function should at least contain five load levels, where three load levels are positioned in the lower half and two load levels are positioned in the upper half related to a range given by maximum load level during operation.

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

C.2.4.3 If DNVGL-ST-C502 has been selected as the basic design standard, also the fatigue design shall follow this standard.

C.2.4.4 In case of using EN 1992-1-1, the fatigue design shall be performed based on Model Code 2010 with the amendments prescribed in [C.2.4.5].

C.2.4.5 For grouted connections with shear keys where the S-N curves shall be chosen according to Model Code 2010 the required design value of the fatigue strength of the grout under compressive loading $f_{cd,fat}$ shall be determined as follows:

$$f_{cd,fat} = 0.85 \cdot \beta_{cc}(t) \cdot f_{ck} \cdot (1 - f_{ck}/400) / \gamma_c$$

with:

f_{ck} = characteristic cylinder compressive strength in N/mm²

γ_c = material factor for grout according to [6.4.1.4]

$\beta_{cc}(t)$ = coefficient for considering the time-dependent strength increase in the grout. Here $\beta_{cc}(t)$ shall not be set larger than 1.0, corresponding to a cyclic initial loading with a grout age ≥ 28 days. In the case of cyclic initial loading at an earlier age of the grout, $\beta_{cc}(t) < 1.0$ shall be determined and taken into account in the analysis.

For the fatigue design of grouted connections according to Model Code 2010 the design fatigue strength of the grout ($f_{cd,fat}$) shall be reduced to 80% $f_{cd,fat}$ to take into account the specific fatigue behavior of the grouted connection (e.g. interaction steel-grout) additionally to the fatigue damage of the grout material. The S-N-curve according to Model Code 2010 is valid only for dry environmental condition of the grouted connections. The influence of water on the grout properties has to be considered for grouted connections under water or in the splash zone. If the grouted connection is exposed to water the allowed number of load cycles (N_i) shall be reduced for each stress block before the related part damage is calculated. For the stress-blocks having stress variations in the compression-compression range N_i shall be raised to the power of 0.8 (i.e. $N_i^{0.8}$) and for the stress-blocks having stress variations in the compression-tension range N_i shall be raised to the power of 0.65 (i.e. $N_i^{0.65}$).

Guidance note:

In general it is sufficient to investigate the maximum compressive strength for three sections (see Figure C-4), taking into account the load direction of each compression strut:

- (a) σ_{c0} : Partial area loading (A_{c0} , near to the shear key)
- (b) σ_{c1} : Theoretical area to accommodate the resistance partial area load ($A_{c1} \leq 3 \cdot A_{c0}$) Geometric boundary condition have to comply with DNVGL-ST-C502 [6.12] or EN 1992-1-1 Section 6.7
- (c) σ_{c2} : Middle of compression strut (A_{c2})

In case the middle of the compression strut is exposed to reversed loading, the number of cycles should be doubled for damage calculation for section (c).

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

If the guidance note is followed, evidence for validation shall be given, e.g. by analytical method described in [C.1].

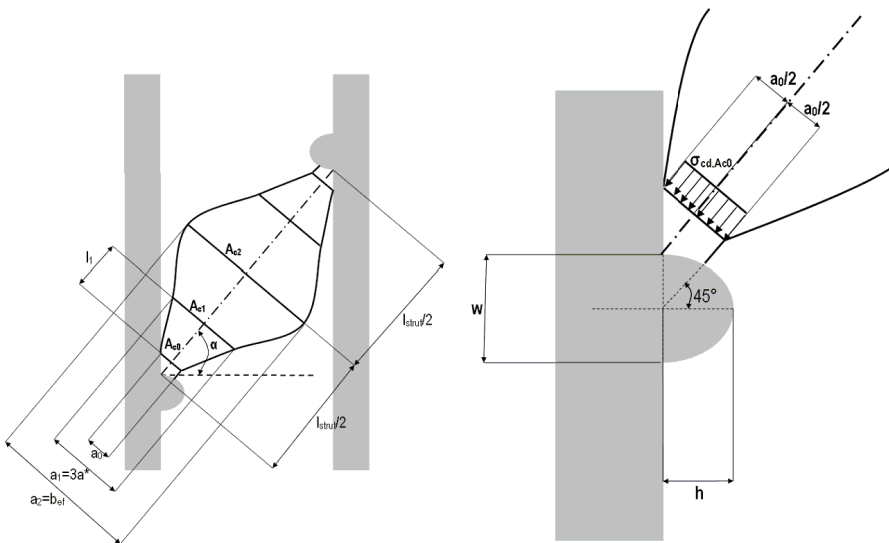


Figure C-4 Compression strut with detail of idealized support area

C.2.5 Fatigue limit states for tubular grouted connections with shear keys - simplified method

C.2.5.1 The FLS verification for the case without stress redistribution and degradation of grout stiffness can be done by simplified fatigue analysis. For this kind of verification only the maximum compressive stress in the grout ($\sigma_{c,max}$) and the total number of load cycles (N_{nom}) are relevant. The method described [5.6.4] shall be followed for this verification.

C.2.5.2 In case the fatigue analysis is performed by [C.2.5] no further damage calculation is required.

C.2.6 Fatigue analysis by consideration of post peak behavior (NV)

C.2.6.1 Various methods considering post peak behavior of grout after virtual cracking are reasonable for verification, in case the material model applied at FEA can be validated by testing.

C.2.7 Fatigue limit states for tubular and conical grouted connections without shear keys

C.2.7.1 Unless data indicate otherwise, the characteristic number of cycles to failure, N , of grout subjected to cyclic stresses can be calculated from:

$$\log_{10} N = C_1 * \left(1 - \frac{\sigma_{max}}{f_{rd}}\right) / \left(1 - \frac{\sigma_{min}}{f_{rd}}\right)$$

where f_{rd} = design reference strength, σ_{max} is the largest value of the maximum principal compressive stress during a stress cycle within the stress block and σ_{min} is the smallest compressive stress in the same direction during this stress cycle. If the direction of the stress is reversed during a stress cycle and the grout is in tension during part of the stress cycle, σ_{min} shall be set equal to zero when the design number of cycles to failure is calculated. If stress cycles occur which keep the grout in tension during the entire cycle, then both σ_{max} and σ_{min} shall be set equal to zero when the design number of cycles to failure is calculated. The factor C_1 shall be taken as:

- 12.0 for structures in air
- 10.0 for structures in water for those stress blocks whose stress variation is in the compression-compression range
- 8.0 for structures in water for those stress blocks whose stress variation is in the compression-tension range.

If the logarithm of the characteristic number of cycles to failure, $\log_{10}N$, calculated according to the expression above, is larger than the value of X given by:

$$X = C_1 / \left(1 - \frac{\sigma_{min}}{f_{rd}} + 0.1 * C_1 \right)$$

then the characteristic number of cycles to failure may be increased further by multiplying the calculated value of $\log_{10}N$ by a factor C_2 which is:

$$C_2 = (1 + 0.2 * (\log_{10} N - X))$$

The design reference strength f_{rd} is calculated as:

$$f_{rd} = C_5 * \frac{f_{cn}}{\gamma_m}$$

where:

- f_{cn} is the nominal compression strength, see [6.3.2.3]
- C_5 is an adjustment factor to obtain a best fit of the above expression for $\log_{10}N$ to experimental fatigue test data from fatigue tests on appropriate grout specimens. Reference is made to DNVGL-ST-C502 [6.13]
- γ_m is the material factor, see [6.4.1].

Guidance note:

The fatigue adjustment factor C_5 can take on values above as well as below 1.0, depending on the grout material. In absence of fatigue tests for grout, C_5 may be taken as 0.8.

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

C.2.7.2 If the structure is designed and analysed according to EN 1992-1-1 reference is made to applicability of sections [5.6.3.7], [5.6.3.8] and [5.6.3.9]. Additionally the design fatigue strength of the grout ($f_{cd,fat}$) shall be reduced to 80% $f_{cd,fat}$ to take into account the specific fatigue behavior of the grouted connection (e.g. interaction steel-grout).

APPENDIX D SCOUR AT A VERTICAL PILE

D.1 Flow around a vertical pile

D.1.1 General

When a vertical pile is placed on a seabed, the water-particle flow associated with currents and passing waves will undergo substantial changes, see [Figure D-1](#). First, a horseshoe vortex will be formed at the base in front of the pile. Second, a vortex flow pattern in the form of vortex shedding will be formed at the lee-side of the pile. Third, the streamlines will contract at the side edges of the pile. This local change in the flow will increase the bed shear stress and the sediment transport capacity will increase accordingly. In the case of an erodible seabed, this may result in a local scour around the pile. Such scour is a threat to the stability of the pile.

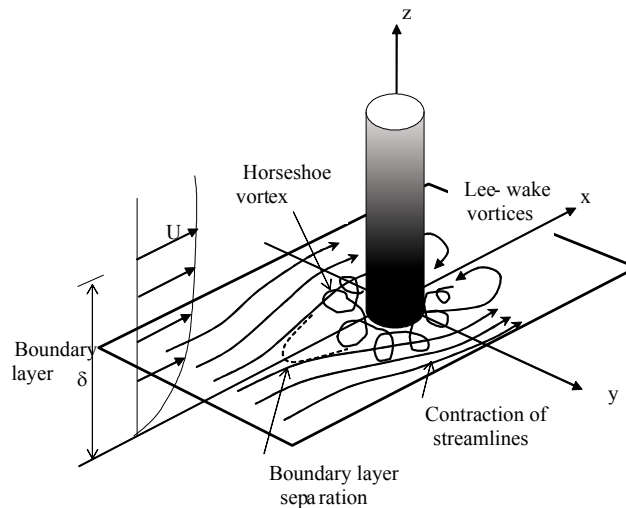


Figure D-1 Flow around the base of a vertical pile

D.2 Bed shear stress

D.2.1 General

D.2.1.1 The increase in the bed shear stress can be expressed in terms of the amplification factor α , which is defined by:

$$\alpha = \frac{\tau_{max}}{\tau_{max,\infty}} \quad (D.1)$$

where τ_{\max} is the maximum value of the bed shear stress τ when the pile structure is present and $\tau_{\max, \infty}$ is the maximum value of the bed shear stress τ_{∞} for the undisturbed flow. In the case of a steady current, τ_{\max} and $\tau_{\max, \infty}$ are replaced by constant τ and τ_{∞} , respectively, in the expression for α .

In the case of a steady current, the amplification factor can become as large as $\alpha = 7-11$. This is due to the presence of a very significant horseshoe vortex. For waves the amplification is smaller.

D.3 Local scour

D.3.1 General

D.3.1.1 The Shields parameter θ is defined by:

$$\theta = \frac{U_f^2}{g(s-1)d} \quad (\text{D.2})$$

where s is the specific gravity of the sediment, d is the grain diameter for the specific grain that will be eroded and U_f is the bed shear velocity. For practical purposes, d_{50} can be used for d , where d_{50} is defined as the median grain diameter in the particle size distribution of the seabed material. The critical Shields parameter, θ_{cr} , is the value of θ at the initiation of sediment motion. The critical value θ_{cr} for the Shields parameter is about 0.05 to 0.06. Seabed erosion starts when the Shields parameter exceeds the critical value.

D.3.1.2 When local scour is analysed, it is important to distinguish between clear-water scour and live-bed scour. This distinction is necessary because the development of a scour hole with time and the relationship between the scour depth and the approach-flow velocity both depend on which of the two types of scour is occurring.

D.3.1.3 Under 'clear water' conditions, i.e. when the sediments far from the pile are not in motion, a state of static equilibrium is reached when the scour hole has developed to an extent such that the flow no longer has the ability to resuspend sediment and remove it from the scour hole. Under 'live bed' conditions, i.e. when the sediment transport prevails over the entire bed, a state of dynamic equilibrium is reached when the rate of removal of material from the scour hole is equal to the rate at which material is being deposited in the scour hole from ambient suspended material and bed loads.

D.3.1.4 In the case of a steady current, the scour process is mainly caused by the presence of the horseshoe vortex combined with the effect of contraction of streamlines at the side edges of the pile, see [Figure D-2](#).

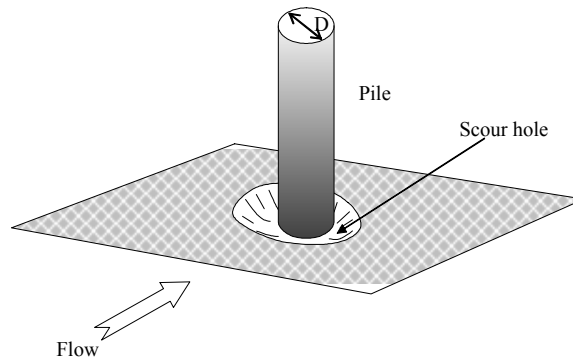


Figure D-2 Scour hole around a vertical pile

D.3.1.5 Field measurements indicate very large variation in scour hole shape. In current dominated sites the scour hole tends to be elongated with steep upstream slope and gentle downstream slope.

D.3.1.6 In the case of waves, the horseshoe vortex and the lee-wake vortex form the two processes that govern the scour. These two processes are primarily governed by the Keulegan-Carpenter number, KC , which is defined by:

$$KC = \frac{u_m \cdot T_p}{D} \quad (D.3)$$

where T_p is the peak wave period, D is the cylinder diameter and $u_m = 1.41u_{rms}$. u_{rms} is the standard deviation of the velocity at the seabed.

D.3.2 Scour depth

D.3.2.1 Unless data, e.g. from model tests, indicate otherwise, the following empirical expression for the equilibrium scour depth, S due to waves may be used:

$$\frac{S}{D} = 1.3 \cdot \{1 - \exp[-0.03(KC - 6)]\} \quad KC \geq 6 \quad (D.4)$$

Caution shall be taken when using this expression, in particular for large-diameter cylinders such as monopiles. The expression is valid for live-bed conditions, i.e. for $\theta > \theta_{cr}$, in which the Shields parameter θ is defined below together with its critical threshold θ_{cr} .

D.3.2.2 For steady current, which implies $KC \rightarrow \infty$, it appears from this expression that $S/D \rightarrow 1.3$.

D.3.2.3 For waves it appears that for $KC < 6$ no scour hole is formed. The physical explanation for this is that no horseshoe vortex develops for $KC < 6$.

D.3.2.4 The equilibrium scour depth S can be used as a basis for structural design. For this purpose the equilibrium scour depth, used as a measure of local scour, may be supplemented with some extra safety margin as appropriate.

D.3.2.5 For steady current the undisturbed bed shear velocity, U_{fc} , is given by the Colebrook and White equation:

$$\frac{U_c}{U_{fc}} = 6,4 - 2,5 \cdot \ln\left(\frac{2,5 * d}{h} + \frac{4,7 * v}{h * U_f}\right) \quad (D.5)$$

where n equal to $10^{-6} \text{ m}^2/\text{s}$ is the kinematic viscosity. The bed shear is related to the bed shear velocity $\tau_c = \rho U_{fc}^2$ where ρ is the water density.

D.3.2.6 For waves, the maximum value of the undisturbed bed shear velocity is calculated by:

$$U_{fw} = \sqrt{\frac{f_w}{2}} \cdot u_m \quad (D.6)$$

where f_w is the coefficient of friction given by:

$$f_w = \begin{cases} 0,04 \cdot (\alpha/k_N)^{-0,25} & \alpha/k_N > 100 \\ 0,04 \cdot (\alpha/k_N)^{-0,75} & \alpha/k_N < 100 \end{cases} \quad (D.7)$$

Here, α is the free stream amplitude, defined by:

$$\alpha = \frac{u_m \cdot T}{2\pi} \quad (D.8)$$

and k_N is the bed roughness equal to $2.5 \cdot d_{50}$, where d_{50} denotes the median grain diameter in the particle size distribution of the seabed material. The bed shear is related to the bed shear velocity $\tau_w = \rho U_{fw}^2$.

D.3.2.7 In combined waves and current the bed shear stress oscillates around a mean value, τ_m , and has maximum value, τ_{max} . The mean and maximum undisturbed combined bed shear stress is calculated as:

$$\tau_{max} = \tau_c \left[1 + 1,2 \left(\frac{\tau_w}{\tau_c + \tau_w} \right)^{3,2} \right] \quad (D.9)$$

$$\rho U_{f, \max}^2 = \tau_{max} = \sqrt{(\tau_m + \tau_w \cos(\Phi))^2 + \tau_w^2 \sin^2(\Phi)} \quad (D.10)$$

where Φ is the angle between wave and current direction.

D.3.3 Time scale of scour


The time development of the scour depth, S , can be expressed as:

$$S_t = S(1 - \exp(-t/T_1)) \quad (D.11)$$

in which t denotes the time, and T_1 denotes the time scale of the scour process. The time scale T_1 of the scour process can be found from the non-dimensional time scale T^* through the following relationship:

$$T^* = \frac{\sqrt{g(s-1)d^3}}{D^2} T_1 \quad (D.12)$$

where T^* is given by the empirical expressions:


$$T^* = \frac{1}{2000} \frac{h}{D} \theta^{-2.2} \quad \text{for steady current} \quad (\text{D.13})$$

$$T^* = 10^{-6} \left(\frac{KC}{\theta} \right)^3 \quad \text{for waves} \quad (\text{D.14})$$

It shall be noted that time scale for backfilling is slower than found by the above equations.

APPENDIX E CALCULATIONS BY FINITE ELEMENT METHOD

E.1 Introduction

E.1.1 General

E.1.1.1 If simple calculations cannot be performed to document the strength and stiffness of a structural component, a finite element analysis should be carried out.

E.1.1.2 The model to be included in the analysis and the type of analysis should be chosen with due consideration to the interaction of the structural component with the rest of the structure.

E.1.1.3 Since a FEM analysis is normally used when simple calculations are insufficient or impossible, care shall be taken to ensure that the model and analysis reflect the physical reality. This shall be done by means of carrying out an evaluation of the input as well as the results from the analysis. Guidelines for such an evaluation are given below.

E.1.1.4 For non-linear FE analysis of steel support structures more guidance can be found in DNVGL-RP-C208.

E.2 Types of analysis

E.2.1 General

Though different types of analyses can be performed by means of FEM analysis, most analyses take the form of static analyses for determination of the strength and stiffness of structures or structural components. FEM analyses are usually computer-based analyses which make use of FEM computer programs.

E.2.2 Static analysis

In a static analysis, structural parts are commonly examined with respect to determining which extreme loads govern the extreme stress, strain and deflection responses. If the analysis is linear, unit loads can be applied, and the response caused by single loads can be calculated. The actual extreme load cases can subsequently be examined by means of linear combinations – superposition.

E.2.3 Eigenfrequency analysis

E.2.3.1 Eigenfrequency analysis is used to determine the eigenfrequencies and normal modes of a structural part.

E.2.3.2 The FEM program will normally perform an analysis on the basis of the lowest frequencies. However, by specifying a shift value, it is possible to obtain results also for a set of higher frequencies around a user-defined frequency.

Guidance note:

The normal modes resulting from a frequency analysis only represent the shape of the deflection profiles, not the actual deflections.

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

E.2.4 Dynamic analysis

Dynamic FEM analysis can be used to determine the time-dependent response of a structural part, e.g. as a transfer function. The analysis is normally based on modal superposition, as this type of analysis is much less time consuming than a 'real' time dependent analysis.

E.2.5 Stability/buckling analysis

E.2.5.1 Stability/buckling analysis is relevant for slender structural parts or sub-parts. This is due to the fact that the loads causing local or global buckling may be lower than the loads causing material strength problems.

E.2.5.2 The analysis is normally performed by applying a set of static loads. Hereafter, the factor by which this set of loads has to be multiplied for stability problems to occur is determined by the analysis program.

E.2.6 Thermal analysis

By thermal analysis, the temperature distribution in structural parts is determined, based on the initial temperature, heat input/output, convection, etc. This is normally a time-dependent analysis; however, it is usually not very time-consuming as only one degree of freedom is present at each modelled node.

Guidance note:

A thermal analysis set-up as described can be used to analyse analogous types of problems involving other time-dependent quantities than temperature. This applies to problems governed by the same differential equation as the one which governs heat transfer. An example of such an application can be found in foundation engineering for analysis of the temporal evolution of settlements in foundation soils.

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

E.2.7 Other types of analyses

E.2.7.1 The analyses listed in [E.2.2] through [E.2.6] only encompass some of the types of analyses that can be performed by FEM analysis. Other types of analyses are: plastic analyses and analyses including geometric non-linearities.

E.2.7.2 Combinations of several analyses can be performed. As examples hereof, the results of an initial frequency analysis can be used as a basis for subsequent dynamic analysis, and the results of a thermal analysis may be used to form a load case in a subsequent static analysis.

E.3 Modelling

E.3.1 General

The results of a FEM analysis are normally documented by plots and printouts of selected extreme response values. However, as the structural FEM model used can be very complex, it is important also to document the model itself. Even minor deviations from the intention may give results that do not reflect reality properly.

E.3.2 Model

The input for an FEM model shall be documented thoroughly by relevant printouts and plots. The printed data should preferably be stored or supplied as files on a CD-ROM.

E.3.3 Coordinate systems

E.3.3.1 Different coordinate systems may be used to define the model and the boundary conditions. Hence the coordinate system valid for the elements and boundary conditions should be checked, e.g. by plots. This is particularly important for beam elements given that it is not always logical which axes are used to define the sectional properties.

E.3.3.2 In a similar manner, as a wrong coordinate system for symmetry conditions may seriously corrupt the results, the boundary conditions should be checked.

E.3.3.3 Insofar as regards laminate elements, the default coordinate system often constitutes an element coordinate system, which may have as a consequence that the fibre directions are distributed randomly across a model.

E.3.4 Material properties

E.3.4.1 Several different material properties may be used across a model, and plots should be checked to verify that the material is distributed correctly.

E.3.4.2 Drawings are often made by means of using units of mm to obtain appropriate values. When the model is transferred to the FEM program, the dimensions are maintained. In this case care should be taken in setting the material properties (and loads) correctly, as kg-mm-N-s is not a consistent set of units. It is advisable to use SI-units (kg-m-N-s).

E.3.5 Material models

The material model used is usually a model for isotropic material, i.e. the same properties prevail in all directions. Note, however, that for composite materials an orthotropic material model has to be used to reflect the different material properties in the different directions. For this model, material properties are defined for three orthogonal directions. By definition of this material, the choice of coordinate system for the elements has to be made carefully.

E.3.6 Elements

For a specific structural part, several different element types and element distributions may be relevant depending on the type of analysis to be carried out. Usually, one particular element type is used for the creation of a FEM model. However, different element types may be combined within the same FEM model. For such a combination special considerations may be necessary.

E.3.7 Element types


E.3.7.1 1D elements consist of for example beam and truss elements:

Models with beam elements are quite simple to create and provide good results for frame structures.

One difficulty may be that the sectional properties are not visible. Hence, the input should be checked carefully for the direction of the section and the numerical values of the sectional properties. Some FEM programs can generate 3D views showing the dimensions of the sections. This facility should be used, if present.

Naturally, the stresses in the connections cannot be calculated accurately by the use of beam elements only.

E.3.7.2 2D elements consist of shell and plate elements:



Shell and plate elements should be used for parts consisting of plates or constant thickness sub-parts. As shell elements suitable for thick plates exist, the wall thickness does not need to be very thin to obtain a good representation by such elements. These elements include the desired behaviour through the thickness of the plate. The same problems as for beam elements are present for shell elements as the thickness of the plates is not shown. The thickness can, however, for most FEM programs be shown by means of colour codes, and for some programs the thickness can be shown by 3D views.

Care shall be taken when reading stress values for shell elements as the upper and lower surface stress distribution might be graphically plotted on the 2D shape (i.e. not considering the thickness of the element). The stresses at connections such as welds cannot be found directly by these elements either.

E.3.7.3 3D elements consist of solid elements:

By the use of solid elements the correct geometry can be modelled to the degree of detail wanted. However, this may imply that the model will include a very large number of nodes and elements, and hence the computation time can be very long. Furthermore, as most solid element types only have three degrees of freedom at each node, the mesh for a solid model may need to be denser than for a beam or shell element model.

E.3.8 Combinations

E.3.8.1 Combination of the three types of elements is possible, however, as the elements may not have the same number of degrees of freedom (DOF) at each node, care should be taken not to create unintended hinges in the model.

E.3.8.2 Beam elements have six degrees of freedom in each node – three translations and three rotations, while solid elements normally only have three – the three translations. Shell elements normally have five degrees of freedom – the rotation around the surface normal is missing. However, these elements may have six degrees of freedom, while the stiffness for the last rotation is fictive.

E.3.8.3 The connection of beam or shell elements to solid elements in a point, respectively a line, introduces a hinge. This problem may be solved by adding additional 'dummy' elements to get the correct connection. Alternatively, constraints may be set up between the surrounding nodal displacements and rotations. Some FEM programs can set up such constraints automatically and the resulting constraint formulations shall in those cases be carefully checked.

E.3.9 Element size and distribution of elements

E.3.9.1 The size, number and distribution of elements required in an actual FEM model depend on the type of analysis to be performed and on the type of elements used.

E.3.9.2 Generally, as beam and shell elements have five or six degrees of freedom in each node, good results can be obtained with a small number of elements. As solid elements only have three degrees of freedom in each node, they tend to be more stiff. Hence, more elements are needed.

E.3.9.3 The shape and order of the elements influence the required number of elements. Triangular elements are more stiff than quadrilateral elements, and first-order elements are more stiff than second-order elements.

E.3.9.4 A mesh sensitivity studies should be carried out as described in [E.4.9] to estimate the actual solutions and FE errors and to ensure the results have converged.

Guidance note:

The required number of elements and its dependency on the element shape are illustrated in an example, in which a cantilever is modelled by beam, membrane, shell and solid elements, see [Figure E-1](#).

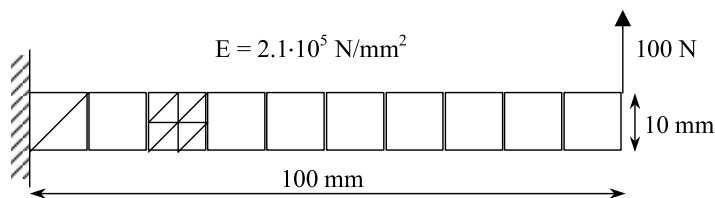


Figure E-1 Cantilever

[Table E-1](#) gives the required number of elements as a function of the element type applied, and the corresponding analysis results in terms of displacements and stresses are also given.

Table E-1 Analysis of cantilever with different types of elements.

Element type	Description	Number of elements	u_y [mm]	$\sigma_{x,node}$ [MPa]	$\sigma_{x,element}$ [MPa]
Analytical result		–	1.9048	600	600
BEAM2D	Beam element, 2 nodes per element, 3 DOF per node, u_x , u_y and θ_z	10	1.9048	600	600
		1	1.9048	600	600
PLANE2D	Membrane element, 4 nodes per element, 2 DOF per node, u_x and u_y	10×1	1.9124	570	0
TRIANG	Membrane element, 3 nodes per element, 2 DOF per node, u_x and u_y	$10 \times 1 \times 2$	0.4402	141	141
		$20 \times 2 \times 2$	1.0316	333	333
		$40 \times 4 \times 2$	1.5750	510	510
SHELL3	Shell element, 3 nodes per element, 6 DOF per node	$20 \times 2 \times 2$	1.7658	578	405
SOLID	Solid element, 8 nodes per element, 3 DOF per node u_x , u_y and u_z	10×1	1.8980	570	570
TETRA4	Solid element, 4 nodes per element, 3 DOF per node u_x , u_y and u_z	$10 \times 1 \times 1$	0.0792	26.7	26.7
		$20 \times 2 \times 1$	0.6326	239	239
		$40 \times 4 \times 1$	1.6011	558	558
TETRA4R	Solid element, 4 nodes per element, 6 DOF per node	$20 \times 2 \times 1$	1.7903	653	487

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

E.3.10 Element quality

E.3.10.1 The results achieved by a certain type and number of elements depend on the quality of the elements. Several measures for the quality of elements can be used; however, the most commonly used are aspect ratio and element warping.

E.3.10.2 The aspect ratio is the ratio between the side lengths of the element. This should ideally be equal to 1, but aspect ratios of up to 3 to 5 do usually not influence the results and are thus acceptable.

E.3.10.3 Element warping is the term used for non-flatness or twist of the elements. Even a slight warping of the elements may influence the results significantly.

E.3.10.4 Most available FEM programs can perform checks of the element quality, and they may even try to improve the element quality by redistribution of the nodes.

E.3.10.5 The quality of the elements should always be checked for an automatically generated mesh, in particular, for the internal nodes and elements. It is usually possible to generate good quality elements for a manually generated mesh.

E.3.10.6 With regard to automatically generated high-order elements, care should be taken to check that the nodes on the element sides are placed on the surface of the model and not just on the linear connection between the corner nodes. This problem often arises when linear elements are used in the initial calculations, and the elements are then changed into higher-order elements for a final calculation.

E.3.10.7 Benchmark tests to check the element quality for different element distributions and load cases are given by NAFEMS. These tests include beam, shell and solid elements, as well as static and dynamic loads.

E.3.11 Boundary conditions

E.3.11.1 The boundary conditions applied to the model should be as realistic as possible. This may require that the FEM model becomes extended to include element models of structural parts other than the particular one to be investigated. One situation where this comes about is when the true supports of a considered structure have stiffness properties which cannot be well-defined unless they are modelled by means of elements that are included in the FEM model.

E.3.11.2 When such an extended FEM model is adopted, deviations from the true stiffness at the boundary of the structural part in question may then become minor only. As a consequence of this, the non-realistic effects due to inadequately modelled boundary conditions become transferred further away to the neighbouring structural parts or sub-parts, which are now represented by elements in the extended FEM model.

E.3.12 Types of restraints

E.3.12.1 The types of restraints normally used are constrained or free displacements/rotations or supporting springs. Other types of restraints may be a fixed non-zero displacement or rotation or a so-called contact, i.e. the displacement is restrained in one direction but not in the opposite direction.

E.3.12.2 The way that a FEM program handles the fixed boundary condition may vary from one program to another. One approach is to remove the actual degree of freedom from the model; another is to apply a spring with a large stiffness at the actual degree of freedom. The latter approach may lead to singularities if the stiffness of the spring is much larger than the stiffness of the element model. Evidently, the stiffness can also be too small, which may in turn compromise the aim for fixation.

An appropriate value for the stiffness of such a stiff spring may be approximately 106 times the largest stiffness of the model.

E.3.12.3 As the program shall first identify whether the displacement has to be constrained or free, the contact boundary condition requires a non-linear calculation.

E.3.13 Symmetry/antimetry

E.3.13.1 Other types of boundary conditions are symmetric and antimetric conditions, which may be applied if the model and the loads possess some kind of symmetry. Taking such symmetry into account may reduce the size of the FEM model significantly.

E.3.13.2 The two types of symmetry that are most frequently used are planar and rotational symmetries. The boundary conditions for these types of symmetry can normally be defined in an easy manner in most FEM programs by using appropriate coordinate systems.

E.3.13.3 The loads for a symmetric model may be a combination of a symmetric and an antimetric load. For linear elastic models this can be considered by calculating the response from the symmetric loads for a model with symmetric boundary conditions, and adding the response from the antimetric loads for a model with antimetric boundary conditions.

E.3.13.4 If both model and loads have rotational symmetry, a sectional model is sufficient for calculating the response.

E.3.13.5 Some FEM programs offer the possibility to calculate the response of a model with rotational symmetry by a sectional model, even if the load is not rotational-symmetric, as the program can model the load in terms of Fourier series.

E.3.14 Loads

E.3.14.1 The loads applied for the FEM calculation are usually structural loads, however, centrifugal loads and temperature loads are also relevant.

E.3.14.2 Structural loads consist of nodal forces and moments and of surface pressure. Nodal forces and moments are easily applied, but may result in unrealistic results locally. This is due to the fact that no true loads act in a single point. Thus, application of loads as pressure loads will in most cases form the most realistic way of load application.

E.3.15 Load application

E.3.15.1 The loading normally consists of several load components, and all of these components may be applied at the same time. As a slightly different load combination in a new analysis will require an entirely new calculation, this is, however, not very rational.

E.3.15.2 For linear elastic modes it is possible to circumvent the problems involved with execution of an entirely new calculation when only a different load combination is considered. Each of the load components should be applied separately as a single load case, and the results found from each of the corresponding analyses should then be combined. In this way, a large range of load combinations can be considered. To facilitate this procedure, unit loads should be used in the single load cases, and the actual loads should then be used in the linear combinations.

E.3.15.3 As only one or more parts of the total structure is modelled, care should be taken to apply the loads as they are experienced by the actual part. To facilitate such load application, 'dummy' elements may be added, i.e. elements with a stiffness representative of the parts which are not modelled – these are often beam elements. The loads can then be applied at the geometrically correct points and be transferred via the beam elements to the structural part being considered.

E.4 Documentation

E.4.1 Model

E.4.1.1 The results of a FEM analysis can be documented by a large number of plots and printouts, which can make it an overwhelming task to find out what has actually been calculated and how the calculations have been carried out.

E.4.1.2 The documentation for the analysis should clearly document which model is considered, and the relevant results should be documented by plots and printouts.

E.4.1.3 The model aspects listed in [E.4.2] through [E.4.7] can and should be checked prior to execution of the FEM analysis.

E.4.2 Geometry control

A verification of the geometric model by a check of the dimensions is an important and often rather simple task. This simple check may reveal if numbers have unintentionally been entered in an incorrect manner.

E.4.3 Mass – volume – centre of gravity

The mass and volume of the model should always be checked. Similarly, the centre of gravity should correspond with the expected value.

E.4.4 Material

Several different materials can be used in the same FEM model. Some of these may be fictitious. This should be checked on the basis of plots showing which material is assigned to each element, and by listing the material properties. Here, care should be taken to check that the material properties are given according to a consistent set of units.

E.4.5 Element type

Several different element types can be used, and here plots and listing of the element types should also be presented.

E.4.6 Local coordinate system


With regard to beam and composite elements, the local coordinate systems should be checked, preferably, by plotting the element coordinate systems.

E.4.7 Loads and boundary conditions

The loads and boundary conditions should be plotted to check the directions of these, and the actual numbers should be checked from listings. To be able to check the correspondence between plots and listings, documentation of node/element numbers and coordinates may be required.

E.4.8 Reactions

E.4.8.1 The reaction forces and moments are normally calculated by the FEM programs and should be properly checked. As a minimum, it should be checked that the total reaction corresponds with the applied



loads. This is especially relevant when loads are applied to areas and volumes, and not merely as discrete point loads. For some programs it is possible to plot the nodal reactions, which can be very illustrative.

E.4.8.2 A major reason for choosing a FEM analysis as the analysis tool for a structure or structural part is that no simple calculation can be applied for the purpose. This implies that there is no simple way to check the results. Instead checks can be carried out to make probable that the results from the FEM analysis are correct.

E.4.9 Mesh refinement

The simplest way of establishing whether the present model or mesh is dense enough is to re-mesh the model with a more dense mesh, and then calculate the differences between analysis results from use of the two meshes. As several meshes may have to be created and tried out, this procedure can, however, be very time-consuming. Moreover, as modelling simplification can induce unrealistic behaviour locally, this procedure may in some cases also result in too dense meshes. Instead, an indication of whether the model or mesh is sufficient would be preferable.

E.4.10 Results

E.4.10.1 Initially, the results should be checked to see if they appear to be realistic. A simple check is made on the basis of an evaluation of the deflection of the component, which should, naturally, reflect the load and boundary conditions applied as well as the stiffness of the component. Also, the stresses on a free surface should be zero.

E.4.10.2 Most commercial FEM programs have some means for calculation of error estimates. Such estimates can be defined in several ways. One of the most commonly used estimates is an estimate of the error in the stress. The estimated 'correct' stress is found by interpolating the stresses by the same interpolation functions as are used for displacements in defining the element stiffness properties.

Another way of getting an indication of stress errors is given by means of comparison of the nodal stresses calculated at a node for each of the elements that are connected to that node. Large variations indicate that the mesh should be more dense.

E.4.10.3 If the results of the analysis are established as linear combinations of the results from single load cases, the load combination factors used should be clearly stated.

E.4.10.4 The global deflection of the structure should be plotted with appropriately scaled deflections. For further evaluation, deflection components could be plotted as contour plots to see the absolute deflections. For models with rotational symmetry, a plot of the deflection relative to a polar coordinate system may be more relevant for evaluation of the results.

E.4.10.5 All components of the stresses are calculated, and it should be possible to plot each component separately to evaluate the calculated stress distribution.

E.4.10.6 The principal stresses should be plotted with an indication of the direction of the stress component, and these directions should be evaluated in relation to the expected distribution.

E.4.10.7 As for the evaluation of the resulting stresses, also the components of the resulting strains and the principal strain should be plotted in an evaluation of the results from the analysis.



CHANGES - HISTORIC

April 2016 edition

Main changes April 2016

General

This document supersedes DNV-OS-J101, May 2014.

This document has been totally revised.

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