Design of Offshore Wind Turbine Structures

MAY 2014

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FOREWORD

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CHANGES – CURRENT

General
This document supersedes DNV-OS-J101, January 2013.

Text affected by the main changes in this edition is highlighted in red colour. However, if the changes involve a whole chapter, section or sub-section, normally only the title will be in red colour.

Det Norske Veritas AS, company registration number 945 748 931, has on 27th November 2013 changed its name to DNV GL AS. For further information, see www.dnvgl.com. Any reference in this document to “Det Norske Veritas AS” or “DNV” shall therefore also be a reference to “DNV GL AS”.

Main changes May 2014

- **General**
  The structure of this document has been converted to decimal numbering. Older references to this document may normally be interpreted by analogy to this example:
  - “DNV-OSS-101 Ch.2 Sec.3 D506” is now “DNV-OSS-101 Ch.2 Sec.3 [4.5.6]”.

- **Sec.1 Introduction**
  - A new item [1.1.5] has been introduced, addressing the issue of equivalence and future developments, allowing for alternative designs and deviations from specified design requirements as long as it can be demonstrated that the safety level is at least as high as that implied by the requirements of the present standard.

- **Sec.3 Site conditions**
  - Two new items [3.3.5.7] and [3.3.5.8], have been included with recommendations for wave theories to be used for regular and irregular waves in deep and shallow waters.
  - In [3.5.1], text about water level has been expanded and Figure 3-10 has been replaced by an improved figure.

- **Sec.4 Loads and load effects**
  - [4.4.5]: Requirements for design of railings have been changed to bring the requirements in alignment with requirements in EN 50308.
  - In [4.2.1.2], a clarification regarding characteristic loads in temporary design conditions has been added.
  - The condition of broadside collision for calculation of operational ship impacts has been removed, to bring the standard in compliance with IEC 61400-3.
  - In [4.5.5], the qualitative requirement for sufficient air gap has been replaced by two quantitative requirements for air gap. The first requirement consists of the former recommendation which has been turned into the requirement [4.5.5.2]. The second requirement is adopted from IEC 61400-3.

- **Sec.5 Load and resistance factors**
  - Table 5-1: clarification regarding application of load factor sets (a) and (b).
  - In [5.2.3], the statement that the material factor in the ALS and SLS shall always be taken as 1.0 has been modified by adding “unless otherwise specified” in order to accommodate exceptions which occur for concrete and grout.

- **Sec.6 Materials**
  - In [6.1.4.12], an option has been included so that a requirement for NV steel grade F can be relaxed to NV grade E when the design temperature is equal to −20°C or higher. This relaxation is in line with intended future changes in requirements for offshore classification.
  - In [6.1.4.14], the requirement for special considerations has been replaced by an option for such considerations, thereby to bring the standard in alignment with DNV-OS-C101.
  - In [6.3], the requirements for grout materials have been modified to comply with requirements in DNV-OS-C502. Some requirements have been replaced by references to DNV-OS-C502.

- **Sec.7 Design of steel structures**
  - The requirement for DFF in atmospheric zone has been changed from 1.0 to 2.0. This is of particular relevance for tower design. DFF = 1.0 implies that regular inspections have to be carried out which is not used much for offshore wind farms. Towers are often designed according to Eurocode 3, which requires DFF = 2.0. The change from DFF = 1.0 to DFF = 2.0 brings DNV-OS-J101 in line with DNV-OS-J103.
An opening to still use DFF = 1.0 is kept provided an inspection plan is established with documented inspection method and inspection intervals that will result in the same safety level as DFF = 2.0 without inspections.

— In [7.3.1.4], the requirement for using a material factor of minimum of 1.2 for global buckling checks refers to IEC 61400-1 and is therefore changed to apply to towers. An extension to also cover monopiles is natural and has been included. An exception is made that a material factor of 1.1 can be used when formulas given in EN1993-1-6 are used for design.

— In [7.8.2.5], the reference to rudders and other ship construction details has been removed.

— Subsection [7.10] has been updated by:
  - retaining only one option for calculation of design fatigue damage by Miner's rule, i.e. the option based on design fatigue factors DFF, thereby to bring the standard in alignment with the format used in ISO19902, which IEC61400-3 refers to for fatigue design
  - introducing a requirement for accounting for system effects in fatigue design when such system effects are present
  - revising the parts dealing with reduction factors for stress range as a function of the mean stress.

— In [7.10.3.5], the provisions for use of stress reduction factors has been restricted to welds which are subjected to PWHT, and the mismatch between reduction factor values in text and figure has been corrected.

— In [7.10.7], the thickness effect exponent $k$ for S-N curve 'C' has been changed from $k = 0.15$ to $k = 0.05$ to be consistent with DNV-RP-C203.

• Sec.8 Detailed design of offshore concrete structures
  — In [8.7.1.3], the text has been revised to bring it in line with updated DNV-OS-C502.
  — In [8.7.2.1], an error in the equation has been corrected.

• Sec.9 Design and construction of grouted connections
  — The section has been expanded by including design rules for grouted connections with shear keys in monopile structures and in jacket structures, based on recent research results.
  — In [9.2.1], “Tubular and conical grouted connections in monopiles without shear keys” has been rearranged.
  — The previous subsection for grouted connections with shear keys, without significant bending moment and subjected to axial load and torque has been removed as obsolete.
  — A new item [9.4.4] “Early age cycling” has been added.

• Sec.10 Foundation design
  — The guidance note in [10.1.3.1] has been updated.
  — The new item [10.3.2.7] regarding permanent buckling and plastic hinges in monopile foundations has been added.

• Sec.11 Corrosion protection
  This section has been thoroughly updated. Amongst the modifications performed the following are mentioned:
  — In [11.1.1.2], a guidance note regarding protection of inert atmosphere has been added.
  — In [11.2.2], the text regarding splash zone definition has been revised for the purpose of clarification and pointed out the difference between the reference wave height used for this definition and the one used for the definition in DNV-OS-C101.
  — The requirements for minimum corrosion allowance have been relaxed by allowing exceptions from specified values when data and experience indicate otherwise.
  — In [11.4.2.4] and [11.4.3.6], relaxed maximum time before survey is carried out from 180 days to 365 days.

• Sec.12 Transport and installation
  — The items [12.4.8.1] and [12.4.8.2] have been added, addressing the logistic challenges involved with the installation of the wind turbine structures in a large wind farm.

• Sec.13 In-service inspection, maintenance and monitoring
  — In [13.2.1.2], the scope for periodical inspection has been changed. Electrical systems above water are no longer included. This also affects detailed survey scope in [13.3.3.1] regarding electrical systems, transformers and generators.
  — In [13.3.5.2], the guidance note regarding calculation of inspection intervals has been updated.
  — In [13.3.7.3], the requirement for measuring the anode potential has been corrected to refer to measurement of the protection potential.

• App.D Stress extrapolation for welds
  — The text has been replaced by a reference to DNV-RP-C203.
• **App.F Pile resistance and load-displacement relationships**
  — The criterion for horizontal-to-vertical force ratio for sliding assessment has been removed on the grounds that it is insufficiently substantiated.
  — In [F.2.4.1], a statement has been included that recommended p-y curves are calibrated for long slender jacket piles, not for large-diameter monopiles.

• **App.G Bearing capacity and stiffness formulae for gravity base foundations**
  — In [G.1], a clarification has been included that all forces referenced in the appendix are factored forces, i.e. they are design forces which have been calculated as characteristic forces multiplied by a load factor.
  — In [G.2], the text explaining the procedure for calculating a rectangular area to represent the effective foundation area of an eccentrically loaded circular foundation has been amended for clarification, and the associated drawing has been replaced by an improved illustration.

**Editorial corrections**

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

**Acknowledgements**

This Offshore Standard makes use of two figures provided by Mærsk Olie og Gas AS. The two figures consist of Figure 7-13 and Figure 7-14 in Sec. 7. Mærsk Olie og Gas AS is gratefully acknowledged for granting DNV permission to use this material.

The standard also makes use of one figure provided by Prof. S.K. Chakrabarti. The figure appears as Figure 3-7 in Sec.3. Prof. Chakrabarti is gratefully acknowledged for granting DNV permission to use this figure.
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SECTION 1 INTRODUCTION

1.1 Introduction

1.1.1 General

1.1.1.1 This offshore standard provides principles, technical requirements and guidance for design, construction and in-service inspection of offshore wind turbine structures, here defined as the support structures and foundations for offshore wind turbines.

1.1.1.2 DNV-OS-J101 is the DNV standard for design of offshore wind turbine structures. The standard covers structural design of offshore wind turbine structures. 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 can be used as a stand-alone document.

1.1.1.3 The standard shall be used for design of bottom-fixed support structures and foundations for offshore wind turbines. The standard shall also be used for design of support structures and foundations for other structures in an offshore wind farm, such as meteorological masts.

1.1.1.4 For design of floating wind turbine structures and their station keeping systems, DNV-OS-J103 applies, but makes extensive reference to DNV-OS-J101 and requirements set forth in DNV-OS-J101.

1.1.1.5 The standard does not cover design of support structures and foundations for substations for wind farms. For design of support structures and foundations for substations, such as converter stations and transformer stations, DNV-OS-J201 applies.

1.1.1.6 The standard does not cover design of wind turbine components such as nacelle, rotor, generator and gearbox. For structural design of rotor blades DNV-DS-J102 applies. For structural design of wind turbine components for which no DNV standard exists, the IEC61400-1 standard applies.

1.1.1.7 The tower, which usually extends from somewhere above the water level to just below the nacelle, is considered a part of the support structure. The structural design of the tower is therefore covered by this standard, regardless of whether a type approval of the tower exists and is to be applied.

Guidance note:
For a type-approved tower, the stiffness of the towers forms part of the basis for the approval. It is important to make sure not to change the weight and stiffness distributions over the height of the tower relative to those assumed for the type approval.

1.1.1.8 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.

1.1.1.9 DNV-OS-J101 is in compliance with IEC61400-3 and IEC61400-22; however, the requirements set forth in DNV-OS-J101 may in some cases be stricter than those set forth in IEC61400-3 and IEC61400-22. Furthermore, DNV-OS-J101 contains DNV's interpretation of the IEC requirements in cases where the IEC requirements need clarification.

Guidance note:
As part of bringing DNV-OS-J101 in compliance with IEC61400-3, an attempt has been made to harmonise DNV-OS-J101 with IEC61400-3, in particular with respect to the specification of load cases. For further information, reference is made to IEC61400-3.

1.1.1.10 DNV-OS-J101 is applied as part of the basis for carrying out a DNV project certification of an offshore wind farm.

1.1.2 Objectives

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

1.1.2.2 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 offshore structures subject to DNV certification
— serve as a basis for verification of offshore wind turbine structures for which DNV is contracted to perform
the verification.

1.1.3 Scope and application

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

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

1.1.3.3 This standard gives requirements for the following:
— design principles
— selection of material and extent of inspection
— design loads
— load effect analyses
— load combinations
— structural design
— foundation design
— corrosion protection.

1.1.4 Non-DNV codes

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

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

1.1.4.3 Where reference in this standard is made to codes other than DNV 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.

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

1.1.4.5 National and governmental regulations may override the requirements of this standard as applicable.

1.1.5 Equivalence and future developments

This standard specifies requirements for the design of offshore wind turbine 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 offshore 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 DNV-RP-A203.

---end---of---Guidance---note---

1.2 Normative references

1.2.1 General

The standards in Table 1-1 include provisions, which through reference in this text constitute provisions of this
standard.

<p>| Table 1-1  DNV Offshore Standards, Rules and Standards for Certification |</p>
<table>
<thead>
<tr>
<th>Reference</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNV-OS-B101</td>
<td>Metallic Materials</td>
</tr>
<tr>
<td>DNV-OS-C101</td>
<td>Design of Offshore Steel Structures, General (LRFD Method)</td>
</tr>
<tr>
<td>DNV-OS-C201</td>
<td>Structural Design of Offshore Units (WSD method)</td>
</tr>
<tr>
<td>DNV-OS-C401</td>
<td>Fabrication and Testing of Offshore Structures</td>
</tr>
<tr>
<td>DNV-OS-C502</td>
<td>Offshore Concrete Structures</td>
</tr>
</tbody>
</table>
1.3 Informative references

1.3.1 General

The documents in Table 1-2, Table 1-3 and Table 1-4 include acceptable methods for fulfilling the requirements in the standards. See also current DNV List of Publications. Other recognised codes or standards may be applied provided it is shown that they meet or exceed the level of safety of the actual standard.

| Table 1-2  DNV Offshore Standards for structural design |
|---|---|
| Reference | Title |
| DNV-OS-C501 | Composite Components |

| Table 1-3  DNV Recommended Practices and Classification Notes |
|---|---|
| Reference | Title |
| DNV/RIso | Guidelines for Design of Wind Turbines |
| DNV-RP-A203 | Technology Qualification |
| DNV-RP-B401 | Cathodic Protection Design |
| DNV-RP-C201 | Buckling Strength of Plated Structures |
| DNV-RP-C202 | Buckling Strength of Shells |
| DNV-RP-C203 | Fatigue Strength Analysis of Offshore Steel Structures |
| DNV-RP-C205 | Environmental Conditions and Environmental Loads |
| DNV-RP-C207 | Statistical Representation of Soil Data |
| DNV-RP-H101 | Risk Management in Marine and Subsea Operations |
| Classification Notes 30.1 | Buckling Strength Analysis of Bars and Frames, and Spherical Shells |
| Classification Notes 30.4 | Foundations |
| Classification Notes 30.6 | Structural Reliability Analysis of Marine Structures |
| Classification Notes 30.7 | Fatigue Assessments of Ship Structures |

| Table 1-4  Other references |
|---|---|
| Reference | Title |
| AISC | LRFD Manual of Steel Construction |
| API RP 2N | Recommended Practice for Planning, Designing, and Constructing Structures and Pipelines for Arctic Conditions |
| BS 7910 | Guide on methods for assessing the acceptability of flaws in fusion welded structures |
| BSH 7004 | Standard Baugrunderkundung. Mindestenanforderungen für Gründungen von Offshore-Windenergieanlagen (Ground Investigations for Offshore Wind Farms) |
| DIN 4131 | Stahlerne Antennentragwerke (Steel radio towers and masts) |
| DIN 4133 | Schornsteine aus Stahl |
### Table 1-4 Other references (Continued)

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>EN 196-1</td>
<td>Methods of testing cement - Part 1: Determination of strength</td>
</tr>
<tr>
<td>EN 10025-1</td>
<td>Hot rolled products of non-alloy structural steels</td>
</tr>
<tr>
<td>EN 10025-2</td>
<td>Hot rolled products of structural steels. Technical delivery conditions for non-alloy structural steels</td>
</tr>
<tr>
<td>EN 10025-3</td>
<td>Hot rolled products of structural steels. Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels</td>
</tr>
<tr>
<td>EN 10025-4</td>
<td>Hot rolled products of structural steels. Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels</td>
</tr>
<tr>
<td>EN 10025-6</td>
<td>Hot rolled products of structural steels. Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition</td>
</tr>
<tr>
<td>EN 10204</td>
<td>Metallic products – types of inspection documents</td>
</tr>
<tr>
<td>EN 10225</td>
<td>Weldable structural steels for fixed offshore structures – technical delivery conditions</td>
</tr>
<tr>
<td>EN 12495</td>
<td>Corrosion Protection of Fixed Offshore Structures</td>
</tr>
<tr>
<td>EN 13670-1</td>
<td>Execution of Concrete Structures – Part 1: Common rules</td>
</tr>
<tr>
<td>EN 1992-1-1</td>
<td>Eurocode 2: Design of Concrete Structures</td>
</tr>
<tr>
<td>EN 1993-1-1</td>
<td>Eurocode 3: Design of Steel Structures, Part 1-1: General Rules and Rules for Buildings</td>
</tr>
<tr>
<td>EN 1993-1-6</td>
<td>Eurocode 3: Design of Steel Structures, Part 1-6: Strength and Stability of Shell Structures</td>
</tr>
<tr>
<td>EN 1997-1</td>
<td>Eurocode 7: Geotechnical Design – Part 1: General rules</td>
</tr>
<tr>
<td>EN 1997-2</td>
<td>Eurocode 7: Geotechnical Design – Part 2: Ground investigation and testing</td>
</tr>
<tr>
<td>EN 1090-1</td>
<td>Execution of steel structures and aluminium structures – Part 1: Requirements for conformity assessment of structural components</td>
</tr>
<tr>
<td>EN 50308</td>
<td>Wind Turbines – Protective measures – Requirements for design, operation and maintenance</td>
</tr>
<tr>
<td>EN 62305</td>
<td>Protection against lightning</td>
</tr>
<tr>
<td>ENV 1090-5</td>
<td>Execution of steel structures – Part 5: Supplementary rules for bridges</td>
</tr>
<tr>
<td>IEC 61400-1</td>
<td>Wind Turbines – Part 1: Design Requirements</td>
</tr>
<tr>
<td>IEC 61400-3</td>
<td>Wind Turbines – Part 3: Design requirements for offshore wind turbines</td>
</tr>
<tr>
<td>IEC 61400-22</td>
<td>Wind Turbines – Part 22: Conformity testing and certification of wind turbines</td>
</tr>
<tr>
<td>ISO 1461</td>
<td>Hot dip galvanized coatings on fabricated iron and steel articles – Specifications and test methods</td>
</tr>
<tr>
<td>ISO 6934</td>
<td>Steel for the prestressing of concrete</td>
</tr>
<tr>
<td>ISO 6935</td>
<td>Steel for the reinforcement of concrete</td>
</tr>
<tr>
<td>ISO 12944</td>
<td>Paints and varnishes – Corrosion protection of steel structures by protective paint systems</td>
</tr>
<tr>
<td>ISO/IEC 17020</td>
<td>Conformity assessment – Requirements for the operation of various types of bodies performing inspection</td>
</tr>
<tr>
<td>ISO/IEC 17025</td>
<td>General requirements for the competence of calibration and testing laboratories</td>
</tr>
<tr>
<td>ISO 19901-2</td>
<td>Petroleum and natural gas industries – Specific requirements for offshore structures – Part 2: Seismic design procedures and criteria</td>
</tr>
<tr>
<td>ISO 19902</td>
<td>Petroleum and Natural Gas Industries – Fixed Steel Offshore Structures</td>
</tr>
<tr>
<td>ISO 19906</td>
<td>Petroleum and Natural Gas Industries – Arctic offshore structures</td>
</tr>
<tr>
<td>ISO 22475-1</td>
<td>Geotechnical investigation and testing – Sampling methods and groundwater measurements – Part 1: Technical principles for execution</td>
</tr>
<tr>
<td>ISO 20340</td>
<td>Paints and varnishes – Performance requirements for protective paint systems for offshore and related structures</td>
</tr>
<tr>
<td>NACE SP0176</td>
<td>Corrosion Control of Submerged Areas of Permanently Installed Steel Offshore Structures Associated With Petroleum Production</td>
</tr>
<tr>
<td>NACE TPC3</td>
<td>Microbiologically Influenced Corrosion and Biofouling in Oilfield Equipment</td>
</tr>
<tr>
<td>NORSOK</td>
<td>M-501 Surface preparation and protective coating</td>
</tr>
<tr>
<td>NORSOK</td>
<td>N-003 Actions and Action Effects</td>
</tr>
<tr>
<td>NORSOK</td>
<td>N-004 Design of Steel Structures</td>
</tr>
<tr>
<td>NORSOK</td>
<td>G-001 Marine Soil Investigations</td>
</tr>
<tr>
<td>NVN 11400-0</td>
<td>Wind turbines – Part 0: Criteria for type certification – Technical Criteria</td>
</tr>
</tbody>
</table>
1.4 Definitions

1.4.1 Verbal forms

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shall</td>
<td>Verbal form used to indicate requirements strictly to be followed in order to conform to the document.</td>
</tr>
<tr>
<td>Should</td>
<td>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.</td>
</tr>
<tr>
<td>May</td>
<td>Verbal form used to indicate a course of action permissible within the limits of the document.</td>
</tr>
</tbody>
</table>

1.4.2 Terms

<table>
<thead>
<tr>
<th>Term</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abnormal load</td>
<td>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.</td>
</tr>
<tr>
<td>Accidental Limit States (ALS)</td>
<td>Ensure that the structure resists accidental loads and maintain integrity and performance of the structure due to local damage or flooding.</td>
</tr>
<tr>
<td>Agreement or by agreement</td>
<td>Unless otherwise indicated, agreed in writing between contractor and purchaser.</td>
</tr>
<tr>
<td>ALARP</td>
<td>As low as reasonably practicable; notation used for risk.</td>
</tr>
<tr>
<td>Atmospheric zone</td>
<td>The external region exposed to atmospheric conditions.</td>
</tr>
<tr>
<td>Cathodic protection</td>
<td>A technique to prevent corrosion of a steel surface by making the surface to be the cathode of an electrochemical cell.</td>
</tr>
<tr>
<td>Characteristic load</td>
<td>The 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.</td>
</tr>
<tr>
<td>Characteristic load effect</td>
<td>The 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.</td>
</tr>
<tr>
<td>Characteristic resistance</td>
<td>The 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.</td>
</tr>
<tr>
<td>Characteristic material strength</td>
<td>The 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.</td>
</tr>
<tr>
<td>Characteristic value</td>
<td>A 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.</td>
</tr>
<tr>
<td>Classification Notes</td>
<td>The classification notes cover proven technology and solutions which are found to represent good practice by DNV, and which represent one alternative for satisfying the requirements stipulated in the DNV Rules or other codes and standards cited by DNV. The classification notes will in the same manner be applicable for fulfilling the requirements in the DNV offshore standards.</td>
</tr>
<tr>
<td>Coating</td>
<td>Metallic, inorganic or organic material applied to steel surfaces for prevention of corrosion.</td>
</tr>
<tr>
<td>Co-directional</td>
<td>Wind and waves acting in the same direction.</td>
</tr>
<tr>
<td>Contractor</td>
<td>A party contractually appointed by the purchaser to fulfil all, or any of, the activities associated with fabrication and testing.</td>
</tr>
<tr>
<td>Corrosion allowance</td>
<td>Extra wall thickness added during design to compensate for any reduction in wall thickness by corrosion (externally and internally) during design life.</td>
</tr>
<tr>
<td>Current</td>
<td>A flow of water past a fixed point and usually represented by a velocity and a direction.</td>
</tr>
<tr>
<td>Cut-in wind speed</td>
<td>Lowest mean wind speed at hub height at which a wind turbine produces power.</td>
</tr>
<tr>
<td>Cut-out wind speed</td>
<td>Highest mean wind speed at hub height at which a wind turbine is designed to produce power.</td>
</tr>
<tr>
<td>Design basis</td>
<td>A 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.</td>
</tr>
<tr>
<td>Design life</td>
<td>The 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.</td>
</tr>
</tbody>
</table>
### Table 1-5 Terms (Continued)

<table>
<thead>
<tr>
<th>Term</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design temperature</td>
<td>The lowest daily mean temperature that the structure may be exposed to during installation and operation.</td>
</tr>
<tr>
<td>Design value</td>
<td>The 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.</td>
</tr>
<tr>
<td>Driving voltage</td>
<td>The difference between closed circuit anode potential and protection potential.</td>
</tr>
<tr>
<td>Environmental state</td>
<td>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.</td>
</tr>
<tr>
<td>Expected loads and response history</td>
<td>Expected load and response history for a specified time period, taking into account the number of load cycles and the resulting load levels and response for each cycle.</td>
</tr>
<tr>
<td>Expected value</td>
<td>The mean value, e.g. the mean value of a load during a specified time period.</td>
</tr>
<tr>
<td>Fatigue</td>
<td>Degradation of the material caused by cyclic loading.</td>
</tr>
<tr>
<td>Fatigue critical</td>
<td>Structure with predicted fatigue life near the design fatigue life.</td>
</tr>
<tr>
<td>Fatigue Limit States (FLS)</td>
<td>Related to the possibility of failure due to the cumulative damage effect of cyclic loading.</td>
</tr>
<tr>
<td>Foundation</td>
<td>The foundation of a support structure for a wind turbine is in this document reckoned as a structural or geotechnical component, or both, extending from the seabed downwards.</td>
</tr>
<tr>
<td>Guidance Note</td>
<td>Information in the standards in order to increase the understanding of the requirements.</td>
</tr>
<tr>
<td>Gust</td>
<td>Sudden and brief increase of the wind speed over its mean value.</td>
</tr>
<tr>
<td>Highest astronomical tide (HAT)</td>
<td>Level of high tide when all harmonic components causing the tide are in phase.</td>
</tr>
<tr>
<td>Hindcast</td>
<td>A method using registered meteorological data to reproduce environmental parameters. Mostly used for reproduction of wave data and wave parameters.</td>
</tr>
<tr>
<td>Hub height</td>
<td>Height of centre of swept area of wind turbine rotor, measured from mean water level (MWL).</td>
</tr>
<tr>
<td>Idling</td>
<td>Condition of a wind turbine, which is rotating slowly and not producing power.</td>
</tr>
<tr>
<td>Independent organisations</td>
<td>Accredited or nationally approved certification bodies.</td>
</tr>
<tr>
<td>Inspection</td>
<td>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.</td>
</tr>
<tr>
<td>Limit State</td>
<td>A state beyond which the structure no longer satisfies the requirements. The following categories of limit states are of relevance for structures. ULS = ultimate limit state; FLS = fatigue limit state; ALS = accidental limit state; SLS = serviceability limit state.</td>
</tr>
<tr>
<td>Load effect</td>
<td>Effect of a single design load or combination of loads on the equipment or system, such as stress, strain, deformation, displacement, motion, etc.</td>
</tr>
<tr>
<td>Lowest astronomical tide (LAT)</td>
<td>Level of low tide when all harmonic components causing the tide are in phase.</td>
</tr>
<tr>
<td>Lowest daily mean temperature</td>
<td>The lowest value of the annual daily mean temperature curve for the area in question. For seasonally restricted service the lowest value within the time of operation applies.</td>
</tr>
<tr>
<td>Lowest waterline</td>
<td>Typical light ballast waterline for ships, transit waterline or inspection waterline for other types of units.</td>
</tr>
<tr>
<td>Mean</td>
<td>Statistical mean over observation period.</td>
</tr>
<tr>
<td>Mean water level (MWL)</td>
<td>Mean still water level, defined as mean level between highest astronomical tide and lowest astronomical tide.</td>
</tr>
<tr>
<td>Mean zero-upcrossing period</td>
<td>Average period between two consecutive zero-upcrossings of ocean waves in a sea state.</td>
</tr>
<tr>
<td>Metocean</td>
<td>Abbreviation of meteorological and oceanographic.</td>
</tr>
<tr>
<td>Non-destructive testing (NDT)</td>
<td>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.</td>
</tr>
<tr>
<td>Object Standard</td>
<td>The standards listed in Table 1-1.</td>
</tr>
<tr>
<td>Offshore Standard</td>
<td>The DNV offshore standards are documents which presents the principles and technical requirements for design of offshore structures. The standards are offered as DNV’s interpretation of engineering practice for general use by the offshore industry for achieving safe structures.</td>
</tr>
<tr>
<td>Offshore wind turbine structure</td>
<td>A structural system consisting of a support structure for an offshore wind turbine and a foundation for the support structure.</td>
</tr>
<tr>
<td>Omni-directional</td>
<td>Wind or waves acting in all directions.</td>
</tr>
<tr>
<td>Term</td>
<td>Explanation</td>
</tr>
<tr>
<td>------------------------------------------</td>
<td>-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Operating conditions</td>
<td>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 seabed, as applicable.</td>
</tr>
<tr>
<td>Parking</td>
<td>The condition to which a wind turbine returns after a normal shutdown. Depending on the construction of the wind turbine, parking refers to the turbine being either in a standstill or an idling condition.</td>
</tr>
<tr>
<td>Partial Safety Factor Method</td>
<td>Method for design where uncertainties in loads are represented by a load factor and uncertainties in strengths are represented by a material factor.</td>
</tr>
<tr>
<td>Pile head</td>
<td>The position along a foundation pile in level with the seabed. This definition applies regardless of whether the pile extends above the seabed.</td>
</tr>
<tr>
<td>Pile length</td>
<td>Length along a pile from pile head to pile tip.</td>
</tr>
<tr>
<td>Pile penetration</td>
<td>Vertical distance from the seabed to the pile tip.</td>
</tr>
<tr>
<td>Potential</td>
<td>The voltage between a submerged metal surface and a reference electrode.</td>
</tr>
<tr>
<td>Purchaser</td>
<td>The 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.</td>
</tr>
<tr>
<td>Qualified welding procedure specification (WPS)</td>
<td>A welding procedure specification, which has been qualified by conforming to one or more qualified WPQRs.</td>
</tr>
<tr>
<td>Rated power</td>
<td>Quantity of power assigned, generally by a manufacturer, for a specified operating condition of a component, device or equipment. For a wind turbine, the rated power is the maximum continuous electrical power output which a wind turbine is designed to achieve under normal operating conditions.</td>
</tr>
<tr>
<td>Rated wind speed</td>
<td>Minimum wind speed at hub height at which a wind turbine’s rated power is achieved in the case of a steady wind without turbulence.</td>
</tr>
<tr>
<td>Recommended Practice (RP)</td>
<td>The recommended practice publications cover proven technology and solutions which have been found by DNV to represent good practice, and which represent one alternative for satisfying the requirements stipulated in the DNV offshore standards or other codes and standards cited by DNV.</td>
</tr>
<tr>
<td>Redundancy</td>
<td>The ability of a component or system to maintain or restore its function when a failure of a member or connection has occurred. Redundancy can be achieved for instance by strengthening or introducing alternative load paths.</td>
</tr>
<tr>
<td>Reference electrode</td>
<td>Electrode with stable open-circuit potential used as reference for potential measurements.</td>
</tr>
<tr>
<td>Refraction</td>
<td>Process by which wave energy is redistributed as a result of changes in the wave propagation velocity caused by variations in the water depth.</td>
</tr>
<tr>
<td>Reliability</td>
<td>The ability of a component or a system to perform its required function without failure during a specified time interval.</td>
</tr>
<tr>
<td>Residual currents</td>
<td>All other components of a current than tidal current.</td>
</tr>
<tr>
<td>Risk</td>
<td>The qualitative or quantitative likelihood of an accidental or unplanned event occurring considered in conjunction with the potential consequences of such a failure. In quantitative terms, risk is the quantified probability of a defined failure mode times its quantified consequence.</td>
</tr>
<tr>
<td>Rotor-nacelle assembly</td>
<td>Part of wind turbine carried by the support structure.</td>
</tr>
<tr>
<td>Scour zone</td>
<td>The external region of the unit which is located at the seabed and which is exposed to scour.</td>
</tr>
<tr>
<td>Serviceability Limit States (SLS)</td>
<td>Imply deformations in excess of tolerance without exceeding the load-carrying capacity, i.e., they correspond to tolerance criteria applicable to normal use.</td>
</tr>
<tr>
<td>Shakedown</td>
<td>A linear elastic structural behaviour is established after yielding of the material has occurred.</td>
</tr>
<tr>
<td>Slamming</td>
<td>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.</td>
</tr>
<tr>
<td>Specified Minimum Yield Strength (SMYS)</td>
<td>The minimum yield strength prescribed by the specification or standard under which the material is purchased.</td>
</tr>
<tr>
<td>Specified value</td>
<td>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.</td>
</tr>
<tr>
<td>Splash zone</td>
<td>External or internal surfaces of a structure which are intermittently wetted by tide or waves or both.</td>
</tr>
<tr>
<td>Standstill</td>
<td>The condition of a wind turbine generator system that is stopped.</td>
</tr>
<tr>
<td>Submerged zone</td>
<td>The part of the installation which is below the splash zone, including the scour zone and permanently buried parts.</td>
</tr>
</tbody>
</table>
1.5 Acronyms, abbreviations and symbols

1.5.1 Acronyms and abbreviations

Acronyms and abbreviations as shown in Table 1-6 are used in this standard.

Table 1-6 Acronyms and abbreviations

<table>
<thead>
<tr>
<th>Short form</th>
<th>In full</th>
</tr>
</thead>
<tbody>
<tr>
<td>AISC</td>
<td>American Institute of Steel Construction</td>
</tr>
<tr>
<td>AISI</td>
<td>American Iron and Steel Institute</td>
</tr>
<tr>
<td>ALARP</td>
<td>As Low As Reasonably Practicable</td>
</tr>
<tr>
<td>ALS</td>
<td>Accidental Limit State</td>
</tr>
<tr>
<td>API</td>
<td>American Petroleum Institute</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
</tr>
<tr>
<td>BS</td>
<td>British Standard (issued by British Standard Institute)</td>
</tr>
<tr>
<td>BSH</td>
<td>Bundesamt für Seeschifffahrt und Hydrographie</td>
</tr>
<tr>
<td>CA</td>
<td>Corrosion Allowance</td>
</tr>
<tr>
<td>CN</td>
<td>Classification Notes</td>
</tr>
<tr>
<td>CP</td>
<td>Cathodic Protection</td>
</tr>
<tr>
<td>CTOD</td>
<td>Crack Tip Opening Displacement</td>
</tr>
<tr>
<td>DDF</td>
<td>Deep Draught Floaters</td>
</tr>
<tr>
<td>DFF</td>
<td>Design Fatigue Factor</td>
</tr>
<tr>
<td>Acronym</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>-------------</td>
</tr>
<tr>
<td>DFT</td>
<td>Dry Film Thickness</td>
</tr>
<tr>
<td>DNV</td>
<td>Det Norske Veritas</td>
</tr>
<tr>
<td>EHS</td>
<td>Extra High Strength</td>
</tr>
<tr>
<td>FLS</td>
<td>Fatigue Limit State</td>
</tr>
<tr>
<td>GACP</td>
<td>Galvanic Anode Cathodic Protection</td>
</tr>
<tr>
<td>HAT</td>
<td>Highest Astronomical Tide</td>
</tr>
<tr>
<td>HAZ</td>
<td>Heat-Affected Zone</td>
</tr>
<tr>
<td>HISC</td>
<td>Hydrogen Induced Stress Cracking</td>
</tr>
<tr>
<td>HS</td>
<td>High Strength</td>
</tr>
<tr>
<td>ICCP</td>
<td>Impressed Current Cathodic Protection</td>
</tr>
<tr>
<td>IEC</td>
<td>International Electrotechnical Commission</td>
</tr>
<tr>
<td>IR</td>
<td>( I \times R = \text{Current} \times \text{Resistance} = \text{Voltage} )</td>
</tr>
<tr>
<td>ISO</td>
<td>International Organization for Standardization</td>
</tr>
<tr>
<td>LAT</td>
<td>Lowest Astronomical Tide</td>
</tr>
<tr>
<td>MP</td>
<td>Monopile</td>
</tr>
<tr>
<td>MWL</td>
<td>Mean Water Level</td>
</tr>
<tr>
<td>MWS</td>
<td>Marine Warranty Survey</td>
</tr>
<tr>
<td>NACE</td>
<td>National Association of Corrosion Engineers</td>
</tr>
<tr>
<td>NDT</td>
<td>Non-Destructive Testing</td>
</tr>
<tr>
<td>NS</td>
<td>Normal Strength</td>
</tr>
<tr>
<td>PWHT</td>
<td>Post Weld Heat Treatment</td>
</tr>
<tr>
<td>RHS</td>
<td>Rectangular Hollow Section</td>
</tr>
<tr>
<td>RMP</td>
<td>Risk Management Plan</td>
</tr>
<tr>
<td>RNA</td>
<td>Rotor-Nacelle Assembly</td>
</tr>
<tr>
<td>ROV</td>
<td>Remotely Operated Vehicle</td>
</tr>
<tr>
<td>RP</td>
<td>Recommended Practice</td>
</tr>
<tr>
<td>SCE</td>
<td>Saturated Calomel Electrode</td>
</tr>
<tr>
<td>SCF</td>
<td>Stress Concentration Factor</td>
</tr>
<tr>
<td>SLS</td>
<td>Serviceability Limit State</td>
</tr>
<tr>
<td>SMYS</td>
<td>Specified Minimum Yield Stress</td>
</tr>
<tr>
<td>SRB</td>
<td>Sulphate Reducing Bacteria</td>
</tr>
<tr>
<td>SWL</td>
<td>Still Water Level</td>
</tr>
<tr>
<td>TLP</td>
<td>Tension Leg Platform</td>
</tr>
<tr>
<td>TP</td>
<td>Transition piece</td>
</tr>
<tr>
<td>ULS</td>
<td>Ultimate Limit State</td>
</tr>
<tr>
<td>WPS</td>
<td>Welding Procedure Specification</td>
</tr>
<tr>
<td>WSD</td>
<td>Working Stress Design</td>
</tr>
</tbody>
</table>
### 1.5.2 Symbols

#### 1.5.2.1 Latin characters

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>a₀</td>
<td>connection area</td>
</tr>
<tr>
<td>b</td>
<td>full breadth of plate flange</td>
</tr>
<tr>
<td>bₑ</td>
<td>effective plate flange width</td>
</tr>
<tr>
<td>c</td>
<td>detail shape factor</td>
</tr>
<tr>
<td>c</td>
<td>wave celerity</td>
</tr>
<tr>
<td>d</td>
<td>bolt diameter</td>
</tr>
<tr>
<td>d</td>
<td>water depth</td>
</tr>
<tr>
<td>f</td>
<td>frequency</td>
</tr>
<tr>
<td>f</td>
<td>load distribution factor</td>
</tr>
<tr>
<td>f₅₈</td>
<td>interface shear capacity in grouted connection</td>
</tr>
<tr>
<td>fₑ₉₈</td>
<td>characteristic compressive strength of 150×300 mm cylinders</td>
</tr>
<tr>
<td>f₉₉</td>
<td>characteristic compressive strength of 75 mm cubes</td>
</tr>
<tr>
<td>f(cn)</td>
<td>normalized compressive strength of concrete</td>
</tr>
<tr>
<td>fₚₑ₇</td>
<td>frequency of ice load</td>
</tr>
<tr>
<td>fₚ₉₇</td>
<td>natural frequency of structure</td>
</tr>
<tr>
<td>fₚₛₗ</td>
<td>strength ratio</td>
</tr>
<tr>
<td>fₚₗ₉</td>
<td>nominal lowest ultimate tensile strength</td>
</tr>
<tr>
<td>fₚₗ₉₉</td>
<td>ultimate tensile strength of bolt</td>
</tr>
<tr>
<td>fₚₗ₃₉</td>
<td>strength ratio</td>
</tr>
<tr>
<td>fₚₗ₉</td>
<td>specified minimum yield stress</td>
</tr>
<tr>
<td>g</td>
<td>acceleration of gravity</td>
</tr>
<tr>
<td>h</td>
<td>height</td>
</tr>
<tr>
<td>h</td>
<td>height of shear keys</td>
</tr>
<tr>
<td>h</td>
<td>water depth</td>
</tr>
<tr>
<td>h₀</td>
<td>reference depth for wind-generated current</td>
</tr>
<tr>
<td>h₉₈</td>
<td>dynamic pressure head due to flow through pipes</td>
</tr>
<tr>
<td>h₉₈₃</td>
<td>vertical distance from the load point to the position of max filling height</td>
</tr>
<tr>
<td>hₙ₉₈</td>
<td>threshold for wave height</td>
</tr>
<tr>
<td>kₙ₉₈</td>
<td>exponent for thickness effects in S-N curves</td>
</tr>
<tr>
<td>kₙ₉₈</td>
<td>number of stress blocks</td>
</tr>
<tr>
<td>kₙ₉ₘ</td>
<td>radial stiffness parameter</td>
</tr>
<tr>
<td>kₙ₉ₘ</td>
<td>wave number</td>
</tr>
<tr>
<td>kₙ₉ₘ</td>
<td>correction factor for aspect ratio of plate field</td>
</tr>
<tr>
<td>kₙ₉ₘ₉</td>
<td>bending moment factor</td>
</tr>
<tr>
<td>kₙ₉ₘ₉₉</td>
<td>fixation parameter for plate</td>
</tr>
<tr>
<td>kₙ₉ₘ₉₉₉</td>
<td>fixation parameter for stiffeners</td>
</tr>
<tr>
<td>kₙ₉ₘ₉₉ₙ</td>
<td>correction factor for curvature perpendicular to the stiffeners</td>
</tr>
<tr>
<td>kₙ₉ₘ₉ₚ₉</td>
<td>support spring stiffness</td>
</tr>
<tr>
<td>kₙ₉ₘ₉ₙ</td>
<td>hole clearance factor</td>
</tr>
<tr>
<td>kₙ₉ₘₙ₉</td>
<td>shear force factor</td>
</tr>
<tr>
<td>l</td>
<td>stiffener span</td>
</tr>
<tr>
<td>lₙ₉₈₉₈</td>
<td>attachment length for welds</td>
</tr>
<tr>
<td>lₙ₉₈₈</td>
<td>elastic length of pile</td>
</tr>
<tr>
<td>lₙ₉₈₉₈₉</td>
<td>distance between points of zero bending moments</td>
</tr>
<tr>
<td>n</td>
<td>number</td>
</tr>
<tr>
<td>p</td>
<td>pressure, nominal pressure</td>
</tr>
<tr>
<td>pₙ₉₈₉₈</td>
<td>design pressure</td>
</tr>
<tr>
<td>pₙ₉₈₉₈₉₉</td>
<td>local pressure</td>
</tr>
<tr>
<td>pₙ₉₈₉₈₉₉₉</td>
<td>nominal pressure</td>
</tr>
<tr>
<td>pₙ₉₈₉₈₉₉₉₉</td>
<td>valve opening pressure</td>
</tr>
<tr>
<td>r</td>
<td>root face</td>
</tr>
<tr>
<td>rₙ₉₈₉₈₉₉₉</td>
<td>radius of curvature</td>
</tr>
<tr>
<td>rₙ₉₈₉₈₉₉₉₉</td>
<td>flexural strength of ice</td>
</tr>
</tbody>
</table>
\( r_{\text{local}} \)  local ice pressure
\( r_u \)  compressive strength of ice
\( s \)  distance between stiffeners
\( s \)  spacing between shear keys
\( s_u \)  undrained shear strength
\( t \)  ice thickness
\( t_0 \)  net thickness of plate
\( t_{\text{eff}} \)  effective thickness
\( t_g \)  thickness of grout
\( t_{IL} \)  thickness of jacket leg
\( t_k \)  corrosion addition
\( t_p \)  wall thickness of pile
\( t_s \)  wall thickness of sleeve
\( t_w \)  throat thickness
\( v_{\text{ave}} \)  annual average wind speed at hub height
\( v_{\text{in}} \)  cut-in wind speed
\( v_{\text{out}} \)  cut-out wind speed
\( v_r \)  rated wind speed
\( v_{\text{tide0}} \)  tidal current at still water level
\( v_{\text{wind0}} \)  wind-driven current at still water level
\( w \)  weld bead width
\( w \)  width of shear keys
\( z \)  vertical distance from still water level, positive upwards
\( z_0 \)  terrain roughness parameter
\( A \)  scale parameter in logarithmic wind speed profile
\( A_C \)  Charnock’s constant
\( A_s \)  net area in the threaded part of a bolt
\( A_W \)  wave amplitude
\( C \)  weld factor
\( C_D \)  drag coefficient
\( C_M \)  mass coefficient
\( C_S \)  slamming coefficient
\( C_e \)  factor for effective plate flange
\( D \)  deformation load
\( D_{IL} \)  diameter of jacket leg
\( D_P \)  diameter of pile
\( D_S \)  diameter of sleeve
\( E \)  modulus of elasticity
\( E \)  environmental load
\( E[] \)  mean value
\( F \)  cumulative distribution function
\( F \)  force, load
\( F_d \)  design load
\( F_k \)  characteristic load
\( F_{pd} \)  design preloading force in bolt
\( F_{H1Shk} \)  tangential (horizontal) force on one vertical shear key
\( F_{V1Shk} \)  vertical force on one shear key
\( G \)  permanent load
\( H \)  height
\( H_{\text{max}} \)  maximum wave height
\( H_0 \)  wave height in deep waters
\( H_{\text{RMS}} \)  root mean squared wave height
\( H_S \)  significant wave height
\( I_p \)  plasticity index (for soil)
\( I_T \)  turbulence intensity
\( I_{\text{ref}} \)  expected turbulence intensity, reference turbulence intensity
K  frost index
KC  Keulegan-Carpenter number
L  length of crack in ice
L  total length of grouted connection, i.e. full length from the grout packers to the outlet holes
Lg  effective length of grouted connection
LS  length of vertical shear key
M  moment
Mp  plastic moment resistance
MT  torque
My  elastic moment resistance
N  fatigue life, i.e. number of cycles to failure
N  number of shear keys
Nd  number of supported stiffeners on the girder span
Ns  number of stiffeners between considered section and nearest support
P  load
P  axial force
Pd  average design point load from stiffeners
Q  variable functional load
R  radius
R  resistance
Rd  design resistance
Rk  characteristic resistance
Rp  outer radius of pile
Rs  outer radius of sleeve
RTP  outer radius of transition piece
S  girder span as if simply supported
SA  response spectral acceleration
SD  response spectral displacement
SV  response spectral velocity
Sd  design load effect
Sk  characteristic load effect
SZl  lower limit of the splash zone
SZu  upper limit of the splash zone
T  wave period
Tc  plate thickness
TD  design useful life of coating
TP  design life of structure
T  peak period
TR  return period
TS  sea state duration
TZ  zero-upcrossing period
U  wind speed, instantaneous wind speed
Uhub  wind speed at hub height
U0  1-hour mean wind speed
U10  10-minute mean wind speed
U10,hub  10-minute mean wind speed at hub height
Uice  velocity of ice floe
V  wind speed
Vcorr  expected maximum corrosion rate
W  steel with improved weldability
Z  steel grade with proven through thickness properties with respect to lamellar tearing.
1.5.2.2 Greek characters

\( \alpha \) angle between the stiffener web plane and the plane perpendicular to the plating
\( \alpha \) exponent in power-law model for wind speed profile
\( \alpha \) coefficient in representation of wave loads according to diffraction theory
\( \alpha \) cone angle of conical grouted connection
\( \alpha \) slope angle of seabed
\( \beta_w \) correlation factor
\( \delta \) deflection
\( \Delta \sigma \) stress range
\( \phi \) friction angle
\( \phi \) resistance factor
\( \gamma \) spectral peak enhancement factor
\( \eta_l \) load factor
\( \eta_m \) material factor
\( \eta_M \) material factor
\( \eta_{Mw} \) material factor for welds
\( \kappa \) Von Karman’s constant
\( \lambda \) wave length
\( \lambda \) reduced slenderness
\( \theta \) rotation angle
\( \mu \) friction coefficient
\( \nu \) Poisson’s ratio
\( \nu \) spectral width parameter
\( \rho \) factor in crack length model for ice
\( \rho \) density
\( \sigma_d \) design stress
\( \sigma_k \) elastic buckling stress
\( \sigma_t \) flexural strength of ice
\( \sigma_{yW} \) characteristic yield stress of weld deposit
\( \sigma_{pd1} \) equivalent design stress for global in-plane membrane stress
\( \sigma_{pd2} \) design bending stress
\( \sigma_{sc} \) characteristic tensile capacity
\( \sigma_{sd} \) design tensile capacity
\( \sigma_U \) standard deviation of wind speed
\( \tau_d \) design shear stress
\( \tau_k \) characteristic shear strength of grouted connection
\( \tau_{kf} \) characteristic interface shear strength due to friction
\( \tau_{kg} \) shear strength of grout
\( \tau_{ks} \) characteristic interface shear strength due to shear keys
\( \tau_{sa} \) shear stress in axially loaded connection
\( \tau_{st} \) shear stress in torsionally loaded connection
\( \omega \) angular frequency
\( \xi \) coefficient in representation of wave loads according to diffraction theory
\( \xi \) empirical coefficient for evaluation of lateral pile spring stiffness in clay
\( \psi \) wake amplification factor
\( \psi \) load combination factor, load reduction factor
\( \psi \) load factor for permanent load and variable functional load
\( \Phi \) standard normal cumulative distribution function.
### 1.5.2.3 Subscripts

- **c**: characteristic value
- **d**: design value
- **k**: characteristic value
- **p**: plastic
- **y**: yield

### 1.6 Support structure concepts

#### 1.6.1 Introduction

**1.6.1.1** Bottom-mounted support structures for large offshore wind farm developments fall into a number of generic types which can be categorised by their nature and configuration, their method of installation, their structural configuration and the selection of their construction materials. The options for offshore support structures basically consist of:

- piled structures
- gravity base structures
- skirt and bucket structures
- moored floating structures.

The structural configuration of support structures can be categorised into five basic types:

- monopile structures
- tripod structures
- lattice structures
- gravity structures
- floating structures.

Hybrid support structure designs may be utilised combining the features of the categorised structures.

Water depth limits proposed for the different types of support structures in the following subsections are meant to be treated as guidance rather than limitations.

**1.6.1.2** Monopile structures provide the benefit of simplicity in fabrication and installation. Tripod and lattice structures are usually piled. Piled foundations by far forms the most common form of offshore foundation. Piled offshore structures have been installed since the late 1940's and have been installed in water depth in excess of 150 metres. The standard method of offshore and near-shore marine installation of piled structures is to lift or float the structure into position and then drive the piles into the seabed using either steam or hydraulic powered hammers. The handling of piles and hammers generally requires the use of a crane with sufficient capacity, ideally a floating crane vessel (revolving or shears leg crane). However, other types of offshore installation units are sometimes used such as drilling jack-ups, specially constructed installation vessels or flat top barges mounted with a land based crawler crane.

**1.6.1.3** Gravity foundations, unlike piled foundations, are designed with the objective of avoiding tensile loads (lifting) between the bottom of the support structure and the seabed. This is achieved by providing sufficient dead loads such that the structure maintains its stability in all environmental conditions solely by means of its own gravity. Gravity structures are usually competitive when the environmental loads are relatively modest and the "natural" dead load is significant or when additional ballast can relatively easily be provided at a modest cost. The ballast can be pumped-in sand, concrete, rock or iron ore. The additional ballast can partly be installed in the fabrication yard and partly at the final position; all depending on the capacity of the construction yard, the available draft during sea transport and the availability of ballast materials. The gravity base structure is especially suited where the installation of the support structure cannot be performed by a heavy lift vessel or other special offshore installation vessels, either because of non-availability or prohibitive costs of mobilising the vessels to the site.

**1.6.1.4** Floating structures can by their very nature be floating directly in a fully commissioned condition from the fabrication and out-fitting yard to the site. Floating structures are especially competitive at large water depths where the depth makes the conventional bottom-supported structures non-competitive.

#### 1.6.2 Gravity base structures and gravity pile structures

**1.6.2.1** The gravity type support structure is a concrete based structure which can be constructed with or without small steel or concrete skirts. The ballast required to obtain sufficient gravity consists of sand, iron ore or rock that is filled into the base of the support structure. The base width can be adjusted to suit the actual soil conditions. The proposed design includes a central steel or concrete shaft for transition to the wind turbine tower. The structure requires a flat base and will for all locations require some form for scour protection, the
extent of which is to be determined during the detailed design.

1.6.2.2 The gravity pile support structure is very much like the gravity support structure. The structure can be filled with iron ore or rock as required. The base width can be adjusted to suit the actual soil conditions. The structure is designed such that the variable loads are shared between gravity and pile actions.

1.6.2.3 These types of structures are well suited for sites with firm soils and water depth ranging from 0 to 25 metres.

1.6.3 Jacket-monopile hybrids and tripods

1.6.3.1 The jacket-monopile hybrid structure is a three-legged jacked structure in the lower section, connected to a monopile in the upper part of the water column, all made of cylindrical steel tubes. The base width and the pile penetration depth can be adjusted to suit the actual soil conditions.

1.6.3.2 The tripod is a standard three-leg structure made of cylindrical 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. Inclined pile sleeves are used when the structure is to be installed with a jack-up drilling rig. The base width and pile penetration depth can be adjusted to suit the actual environmental and soil conditions.

1.6.3.3 These types of structures are well suited for sites with water depth ranging from 20 to 50 metres.

1.6.4 Monopiles

1.6.4.1 The monopile support structure is a simple design by which the tower is supported by the monopile, either directly or through a transition piece, which is a transitional section between the tower and the monopile. The monopile continues down into the soil. The structure is made of cylindrical steel tubes.

1.6.4.2 The pile penetration depth can be adjusted to suit the actual environmental and soil conditions. The monopile is advantageous in areas with movable seabed and scour. A possible disadvantage is a too high flexibility in deep waters. The limiting condition of this type of support structure is the overall deflection and vibration.

1.6.4.3 This type of structure is well suited for sites with water depth ranging from 0 to 25 metres.

1.6.5 Supported monopiles and guyed towers

1.6.5.1 The supported monopile structure is a standard monopile supported by two beams piled into the soil at a distance from the monopile. The structure is made of cylindrical steel tubes. The pile penetration of the supporting piles can be adjusted to suit the actual environmental and soil conditions.

1.6.5.2 The guyed tower support structure is a monotower connected to a double hinge near the seabed and allowed to move freely. The tower is supported in four directions by guy wires extending from the tower (above water level) to anchors in the seabed. The support structure installation requires use of small to relatively large offshore vessels. Anchors including mud mats are installed. Guy wires are installed and secured to floaters. Seabed support is installed and the tower is landed. Guy wires are connected to tensioning system. Scour protection is installed as required.

1.6.5.3 These types of structures are well suited for sites with water depth ranging from 20 to 40 metres.

1.6.6 Tripods with buckets

1.6.6.1 The tripod with buckets is a tripod structure equipped with suction bucket anchors instead of piles as for the conventional tripod structure. The wind turbine support structure can be transported afloat to the site. During installation, each bucket can be emptied in a controlled manner, thus avoiding the use of heavy lift equipment. Further, the use of the suction buckets eliminates the need for pile driving of piles as required for the conventional tripod support structure.

1.6.6.2 The support structure shall be installed at locations, which allow for the suction anchor to penetrate the prevalent soils (sand or clay) and which are not prone to significant scour.

1.6.6.3 This type of structure is well suited for sites with water depth ranging from 20 to 50 metres.

1.6.7 Suction buckets

1.6.7.1 The suction bucket steel structure consists of a centre column connected to a steel bucket through flange-reinforced shear panels, which distribute the loads from the centre column to the edge of the bucket. The wind turbine tower is connected to the centre tubular above mean sea level. The steel bucket consists of vertical steel skirts extending down from a horizontal base resting on the soil surface.

1.6.7.2 The bucket is installed by means of suction and will in the permanent case behave as a gravity 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.
1.6.7.3 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 wave, a cavity will tend to develop between the soil surface and the top of the bucket at the heel. 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. The concept allows for a simple decommissioning procedure.

1.6.7.4 This type of structure is well suited for sites with water depth ranging from 0 to 25 metres.

1.6.8 Lattice towers

1.6.8.1 The three-legged lattice tower consists of three corner piles interconnected with bracings. At the seabed pile sleeves are mounted to the corner piles. The soil piles are driven inside the pile sleeves to the desired depth to gain adequate stability of the structure.

1.6.8.2 This type of structure is well suited for sites with water depth ranging from 20 to 40 metres.

1.6.9 Low-roll floaters

1.6.9.1 The low-roll floater is basically a floater kept in position by mooring chains and anchors. In addition to keeping the floater in place, the chains have the advantage that they contribute to dampen the motions of the floater. At the bottom of the hull of the floater, a stabiliser is placed to further reduce roll.

1.6.9.2 The installation is simple since the structure can be towed to the site and then be connected by the chains to the anchors. The anchors can be fluke anchors, drag-in plate anchors and other plate anchors, suction anchors or pile anchors, depending on the actual seabed conditions. When the anchors have been installed, the chains can be installed and tightened and hook-up cables can be installed.

1.6.9.3 This structure is a feasible solution in large water depths.

1.6.10 Tension leg platforms

1.6.10.1 The tension leg support platform is a floater submerged by means of tensioned vertical anchor legs. The base structure helps dampen the motions of the structural system. The installation is simple since the structure can be towed to the site and then be connected to the anchors. When anchors such as anchor piles have been installed and steel legs have been put in place, the hook-up cable can be installed. The platform is subsequently lowered by use of ballast tanks and/or tension systems.

1.6.10.2 The entire structure can be disconnected from the tension legs and floated to shore in case of major maintenance or repair of the wind turbine.

1.6.10.3 This structure is a feasible solution in large water depths.
SECTION 2 DESIGN PRINCIPLES

2.1 Introduction

2.1.1 General

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

— design by partial safety factor method with linear combination of loads or load effects
— design by partial safety factor method with direct simulation of combined load effect of simultaneous load processes
— design assisted by testing
— probability-based design.

2.1.1.2 General design considerations regardless of design method are also given in [2.2.1.1].

2.1.1.3 This standard is 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 allows for 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 and construction 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.

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

2.2.1.5 Structural elements may be manufactured according to the requirements given in DNV-OS-C401.

2.3 Safety classes and target safety level

2.3.1 Safety classes

2.3.1.1 In this standard, structural safety is ensured by use of a safety class methodology. The structure to be designed is classified into a safety class based on the failure consequences. The classification is normally determined by the purpose of the structure. For each safety class, a target safety level is defined in terms of a
nominal annual probability of failure.

2.3.1.2 For structures in offshore wind farms, three safety classes are considered. Low safety class is used for structures, whose failures imply low risk for personal injuries and pollution, low risk for economical consequences and negligible risk to human life. Normal safety class is used for structures, whose failures imply some risk for personal injuries, pollution or minor societal losses, or possibility of significant economic consequences. High safety class is used for structures, whose failures imply large possibilities for personal injuries or fatalities, for significant pollution or major societal losses, or very large economic consequences.

**Guidance note:**
Support structures and foundations for wind turbines, which are normally unmanned, are usually to be designed to the normal safety class. Also support structures and foundations for meteorological measuring masts are usually to be designed to the normal safety class.

Note, however, that the possibility of designing these support structures and foundations to a different safety class than the normal safety class should always be considered, based on economical motivations and considerations about human safety.

For example, the design of a meteorological measuring mast for a large wind farm may need to be carried out to the high safety class, because a loss of the mast may cause a delay in the completion of the wind farm or it may imply overdesign of the turbines and support structures in the wind farm owing to the implied incomplete knowledge of the wind. The costs associated with the loss of such a mast may well exceed the costs associated with the loss of a turbine and thereby call for design to the high safety class.

Also, in order to protect the investments in a wind farm, it may be wise to design the support structures and foundations for the wind turbines to high safety class.

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2.3.1.3 In this standard, the different safety classes applicable for different types of structures are reflected in different requirements to load factors. The requirements to material factors remain unchanged regardless of which safety class is applicable for a structure in question.

2.3.2 Target safety

2.3.2.1 The target safety level for structural design of support structures and foundations for wind turbines to the normal safety class according to this standard is a nominal annual probability of failure of $10^{-4}$. This target safety is the level aimed at for structures, whose failures are ductile, and which have some reserve capacity.

**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.

The target safety level of $10^{-4}$ represents DNV's interpretation of the safety level inherent in the normal safety class for wind turbines defined in IEC61400-1.

The target safety level of $10^{-4}$ is compatible with the safety level implied by DNV-OS-C101 for unmanned structures. This reflects that wind turbines and wind turbine structures designed to normal safety class according to this standard are unmanned structures. For wind turbines where personnel are planned to be present during severe loading conditions, design to high safety class with a nominal annual probability of failure of $10^{-5}$ is warranted.

Structural components and details should be shaped such that the structure as far as possible will behave in the presumed ductile manner. Connections should be designed with smooth transitions and proper alignment of elements. Stress concentrations should be avoided as far as possible. A structure or a structural component may behave as brittle, even if it is made of ductile materials, for example when there are sudden changes in section properties.

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2.3.2.2 The target safety level is the same, regardless of which design philosophy is applied.

**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. Connections should be designed with smooth transitions and proper alignment of elements. Stress concentrations should be avoided as far as possible. A structure or a structural component may behave as brittle, even if it is made of ductile materials, for example when there are sudden changes in section properties.

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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
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 cyclic 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:

**Ultimate limit states (ULS)**
- loss of structural resistance (excessive yielding and buckling)
- failure of components due to brittle fracture
- loss of static equilibrium of the structure, or of a part of the structure, considered as a rigid body, e.g. overturning or capsizing
- failure of critical components of the structure caused by exceeding the ultimate resistance (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).

**Fatigue limit states (FLS)**
- cumulative damage due to repeated loads.

**Accidental limit states (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).

**Serviceability limit states (SLS)**
- deflections that may alter the effect of the acting forces
- deformations that may change the distribution of loads between supported rigid objects and the supporting structure
- excessive vibrations producing discomfort or affecting non-structural components
- motions that exceed the limitation of equipment
- 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 $S$ can 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.

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 $\gamma_{fi}$

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

   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 $\gamma_{fi}$

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

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, including the tower, from the wind loading on the turbine, whereas Approach (2) typically applies to the design of the support structure and foundation with the load effects in the tower applied as a boundary condition.

Guidance note:
For structural design of monopiles and other piled structures, Approach (2) can be used to properly account for the influence from the nonlinearities of the soil. In a typical design situation for a monopile, the main loads will be wind loads and wave loads in addition to permanent loads. The design combined wind and wave load effects at an appropriate interface level, such as the tower flange, can be determined from an integrated structural analysis of the tower and support structure by Approach (1) and consist of a shear force in combination with a bending moment. These design load effects can then be applied as external design loads at the chosen interface level, and the design load effects in the monopile structure and foundation pile for these design loads can then be determined from a structural analysis of the monopile structure and foundation pile by Approach (2).

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 $F_i$, $i = 1,...,n$. It may be expressed as

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

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 $F_i$, $i = 1,...,n$, can 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 $F_i$, $i = 1,...,n$, can 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.

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 $F_i$, $i = 1,...,n$, can be achieved as

$$S_k = \sum_{i=1}^{n} S_{ki}$$
2.5.2.4 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. For further details see [4.6].

2.5.2.5 In this standard, design in the ULS is either 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 $(S_{k1},...,S_{kn})$ 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 $(S_{k1},...,S_{kn})$, 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.

2.5.2.6 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 $\gamma_k$. 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 the 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.

2.5.2.7 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.

2.5.2.8 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 $\gamma_m$:

\[
R_d = \frac{R_k}{\gamma_m}
\]

2) The design resistance $R_d$ is obtained from the design material strength $\sigma_d$ 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 $\sigma_d$ is obtained by dividing the characteristic material strength $\sigma_k$ by a material factor $\gamma_m$.

\[
\sigma_d = \frac{\sigma_k}{\gamma_m}
\]

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.

2.5.2.9 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 $\sigma_k$ is obtained as a specific quantile in the probability distribution of the material strength and may be obtained by testing.

2.5.2.10 Load factors account for:
— possible unfavourable deviations of the loads from their characteristic values
— the limited probability that different loads exceed their respective characteristic values simultaneously
— uncertainties in the model and analysis used for determination of load effects.

2.5.2.11 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 Sec.4.

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

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

2.5.5 Load and resistance factors

Load and resistance factors for the various limit states are given in Sec.5.

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

2.6.1 General

2.6.1.1 Design by direct simulation of the combined load effect of simultaneously acting load processes is similar to design by the partial safety factor method, except that 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.

2.6.1.2 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 may prove an attractive alternative to 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. 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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2.6.2 Design format

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 \( \gamma_f \),

\[ S_d = \gamma_f S_k \]

2.6.3 Characteristic load effect

2.6.3.1 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 must be established, e.g. in the form of a beam-element based frame model, to which loads from several simultaneously acting load processes can 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 can 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.

Further guidance on how to determine the 98% quantile in the distribution of the annual maximum load effect is provided in Sec.4.

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2.6.4 Characteristic resistance

The characteristic resistance is to be calculated as for the partial safety factor method.

2.7 Design assisted by testing

2.7.1 General

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

2.7.1.2 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.

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

2.7.2 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.
2.8 Probability-based design

2.8.1 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.

2.8.2 General

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

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

2.8.2.3 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.

2.8.2.4 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 the standards)
— special case design problems
— novel designs for which limited or no experience exists.

2.8.2.5 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.

2.8.2.6 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, should be agreed in each separate case. To the extent possible, the minimum target reliabilities shall be based on established cases that are known to have adequate safety.

2.8.2.7 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
— Classification Notes No. 30.6.
SECTION 3 SITE CONDITIONS

3.1 Introduction

3.1.1 Definition

3.1.1.1 Site conditions consist of all site-specific conditions which may influence the design of a wind turbine structure by governing its loading, its capacity or both.

3.1.1.2 Site conditions cover virtually all environmental conditions on the site, including but not limited to meteorological conditions, oceanographic conditions, soil conditions, seismicity, biology, and various human activities.

Guidance note:
The meteorological and oceanographic conditions which may influence the design of a wind turbine structure consist of phenomena such as wind, waves, current and water level. These phenomena may be mutually dependent and for the three first of them the respective directions are part of the conditions that may govern the design.

Micro-siting of the wind turbines within a wind farm requires that local wake effects from adjacent wind turbines be considered part of the site conditions at each individual wind turbine structure in the farm.

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3.2 Wind climate

3.2.1 Wind conditions

3.2.1.1 For representation of wind climate, a distinction is made between normal wind conditions and extreme wind conditions. The normal wind conditions generally concern recurrent structural loading conditions, while the extreme wind conditions represent rare external design conditions. Normal wind conditions are used as basis for determination of primarily fatigue loads, but also extreme loads from extrapolation of normal operation loads. Extreme wind conditions are wind conditions that can lead to extreme loads in the components of the wind turbine and in the support structure and the foundation.

3.2.1.2 The normal wind conditions are specified in terms of an air density, a long-term distribution of the 10-minute mean wind speed, a wind shear in terms of a gradient in the mean wind speed with respect to height above the sea surface, and turbulence.

3.2.1.3 The extreme wind conditions are specified in terms of an air density in conjunction with prescribed wind events. The extreme wind conditions include wind shear events, as well as peak wind speeds due to storms, extreme turbulence, and rapid extreme changes in wind speed and direction.

3.2.1.4 The normal wind conditions and the extreme wind conditions shall be taken in accordance with IEC61400-1.

Guidance note:
The normal wind conditions and the extreme wind conditions, specified in IEC61400-1 and used herein, may be insufficient for representation of special conditions experienced in tropical storms such as hurricanes, cyclones and typhoons.

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3.2.2 Parameters for normal wind conditions

3.2.2.1 The wind climate is represented by the 10-minute mean wind speed \( U_{10} \) and the standard deviation \( \sigma_U \) of the wind speed. In the short term, i.e. over a 10-minute period, stationary wind conditions with constant \( U_{10} \) and constant \( \sigma_U \) are assumed to prevail.

Guidance note:
The 10-minute mean wind speed \( U_{10} \) is a measure of the intensity of the wind. The standard deviation \( \sigma_U \) is a measure of the variability of the wind speed about the mean.

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3.2.2.2 The arbitrary wind speed under stationary 10-minute conditions in the short term follows a probability distribution whose mean value is \( U_{10} \) and whose standard deviation is \( \sigma_U \).

3.2.2.3 The turbulence intensity is defined as the ratio \( \sigma_U/U_{10} \).

3.2.2.4 The short term 10-minute stationary wind climate may be represented by a wind spectrum, i.e. the power spectral density function of the wind speed process, \( S(f) \). \( S(f) \) is a function of \( U_{10} \) and \( \sigma_U \) and expresses how the energy of the wind speed is distributed between various frequencies.
3.2.3 Wind data

3.2.3.1 Wind speed statistics are to be used as a basis for representation of the long-term and short-term wind conditions. Empirical statistical data used as a basis for design must cover a sufficiently long period of time, preferably 10 years or more.

**Guidance note:**
Site-specific measured wind data over sufficiently long periods with minimum or no gaps are to be sought. What is a sufficiently long period in this context depends on which kinds of wind parameters are sought. Representativeness of the data for interpretation of the sought-after wind parameters is the key issue. For proper estimation of extreme values many years of data is a must, and generally the length of time covered by data should be long enough that sought-after key figures can be captured and extracted. For example, the mean value of the 10-minute mean wind speed is expected to exhibit variability from year to year such that several years of data are needed in order to get a proper grip on this parameter. For other types of data shorter lengths of time covered by data may suffice, for example turbulence conditioned on 10-minute mean wind speed and standard deviation of wind speed, and standard deviation conditioned on 10-minute mean wind speed.

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3.2.3.2 Wind speed data are height-dependent. The mean wind speed at the hub height of the wind turbine shall be used as a reference. When wind speed data for other heights than the reference height are not available, the wind speeds in these heights can be calculated from the wind speeds in the reference height in conjunction with a wind speed profile above the still water level.

3.2.3.3 The long-term distributions of \( U_{10} \) and \( \sigma_U \) should preferably be based on statistical data for the same averaging period for the wind speed as the averaging period which is used for the determination of loads. If a different averaging period than 10 minutes is used for the determination of loads, the wind data may be converted by application of appropriate gust factors. The short-term distribution of the arbitrary wind speed itself is conditional on \( U_{10} \) and \( \sigma_U \).

**Guidance note:**
An appropriate gust factor to convert wind statistics from other averaging periods than 10 minutes depends on the frequency location of a spectral gap, when such a gap is present. Application of a fixed gust factor, which is independent of the frequency location of a spectral gap, can lead to erroneous results.

The latest insights for wind profiles above water should be considered for conversion of wind speed data between different reference heights or different averaging periods.

Unless data indicate otherwise, the following expression can be used for calculation of the mean wind speed \( U \) with averaging period \( T \) at height \( z \) above sea level as

\[
U(T, z) = U_{10} \cdot (1 + 0.137 \ln \frac{z}{h} - 0.047 \ln \frac{T}{T_0})
\]

where \( h = 10 \text{ m} \) and \( T_{10} = 10 \text{ minutes} \), and where \( U_{10} \) is the 10-minute mean wind speed at height \( h \). This expression converts mean wind speeds between different averaging periods. When \( T > T_{10} \), the expression provides the most likely largest mean wind speed over the specified averaging period \( T \), given the original 10-minute averaging period with stationary conditions and given the specified 10-minute mean wind speed \( U_{10} \). The conversion does not preserve the return period associated with \( U_{10} \). The expression is representative for North Sea conditions.

For extreme mean wind speeds corresponding to specified return periods in excess of approximately 50 years, the following expression can be used for conversion of the one-hour mean wind speed \( U_0 \) at height \( h \) above sea level to the mean wind speed \( U \) with averaging period \( T \) at height \( z \) above sea level

\[
U(T, z) = U_0 \cdot \left[1 + C \cdot \ln \frac{z}{h}\right] \cdot \left[1 - 0.41 \cdot I_U(z) \cdot \ln \frac{T}{T_0}\right]
\]

where \( h = 10 \text{ m}, T_0 = 1 \text{ hour} \) and \( T < T_0 \) and where

\[
C = 5.73 \cdot 10^{-2} \sqrt{1 + 0.15U_0}
\]

and

\[
I_U = 0.06 \cdot (1 + 0.043U_0) \cdot \left(\frac{z}{h}\right)^{-0.22}
\]

and where \( U \) will have the same return period as \( U_0 \).

This conversion expression is recognised as the Frøya wind profile. More details can be found in DNV-RP-C205. Both conversion expressions are based on data from North Sea and Norwegian Sea locations and may not necessarily lend themselves for use at other offshore locations. The expressions should not be extrapolated for use beyond the height range for which they are calibrated, i.e. they should not be used for heights above approximately 100 m. Possible influences from geostrophic winds down to about 100 m height emphasise the importance of observing this restriction.
Both expressions are based on the application of a logarithmic wind profile. For locations where an exponential wind profile is used or prescribed, the expressions should be considered used only for conversions between different averaging periods at a height $z$ equal to the reference height $h = 10$ m.

---end---of---Guidance---note---

3.2.3.4 Empirical statistical wind data used as a basis for design must cover a sufficiently long period of time.

Guidance note:

Wind speed data for the long-term determination of the 10-minute mean wind speed $U_{10}$ are usually available for power output prediction. Turbulence data are usually more difficult to establish, in particular because of wake effects from adjacent operating wind turbines. The latest insights for wind profiles within wind farms should be considered.

---end---of---Guidance---note---

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

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

3.2.3.7 Characteristic values of the wind velocity shall be determined with due account of wake effects owing to the presence of other wind turbines upstream, such as in a wind farm.

Guidance note:

A wind farm generates its own wind climate due to downstream wake effects, and the wind climate in the centre of the wind farm may therefore be very different from the ambient wind climate. The layout of the wind farm has an impact on the wind at the individual wind turbines. Wake effects in a wind farm will in general imply a considerably increased turbulence, reflected in an increased standard deviation $\sigma_U$ of the wind speed. This effect may be significant even when the spacing between the wind turbines in the wind farm is as large as 8 to 10 rotor diameters. Wake effects in a wind farm may also imply a reduction in the 10-minute mean wind speed $U_{10}$ relative to that of the ambient wind climate.

Wake effects in wind farms will often dominate the fatigue loads in offshore wind turbine structures. Wake effects fade out more slowly and over longer distances offshore than they do over land.

For assessment of wake effects in wind farms, the effects of changed wind turbine positions within specified installation tolerances for the wind turbines relative to their planned positions should be evaluated.

Information about wake effects in wind farms is given in IEC61400-1, Annex D.

---end---of---Guidance---note---

3.2.3.8 Wind speed data are usually specified for a specific reference temperature. When wind speed data are used for structural design, it is important to be aware of this reference temperature, in particular with a view to the operation philosophy adopted for the wind turbine design and the temperature assumptions made in this context.

Guidance note:

The wind load on a wind turbine tower is induced by the wind pressure which depends both on density and wind speed. The wind load on the rotor does not depend on the wind pressure alone but also on stall characteristics of the blade profile and active control of the blade pitch and the rotor speed. Design loads in type certification normally refer to an air density of $1.225 \text{ kg/m}^3$. Project specific design loads shall address the air density observed with the wind speed measurements in a rational manner. The air density can increase by up to 10% in arctic areas during the winter season.

---end---of---Guidance---note---

3.2.4 Wind modelling

3.2.4.1 The spectral density of the wind speed process expresses how the energy of the wind turbulence is distributed between different frequencies. The spectral density of the wind speed process including wake effects from any upstream wind turbines is ultimately of interest.

Guidance note:

The latest insights for wind spectrum modelling within wind farms should be considered when the spectral density of the wind speed process is to be established.

---end---of---Guidance---note---

3.2.4.2 Site-specific spectral densities of the wind speed process can be determined from available measured wind data. When measured wind data are insufficient to establish site-specific spectral densities, it is recommended to use a spectral density model which fulfils that the spectral density $S_U(f)$ asymptotically
approaches the following form as the frequency $f$ in the inertial subrange increases:

$$S_U(f) = 0.202\sigma_U^2\left(\frac{L_k}{U_{10}}\right)^2 f^{-5/3}$$

### 3.2.4.3 Unless data indicate otherwise, the spectral density of the wind speed process may be represented by the Kaimal spectrum,

$$S_U(f) = \sigma_U^2 \frac{4L_k}{U_{10}} \left(1 + 6\frac{L_k}{U_{10}}\right)^{-2/3}$$

in which $f$ denotes frequency, and the integral scale parameter $L_k$ is to be taken as

\[
L_z = \begin{cases} 
5.67z & \text{for } z < 60 \text{ m} \\
340.2 \text{ m} & \text{for } z \geq 60 \text{ m}
\end{cases}
\]

where $z$ denotes the height above the seawater level. This model spectrum fulfils the requirement in [3.2.4.2]. Other model spectra for wind speed processes than the Kaimal spectrum can be found in DNV-RP-C205.

**Guidance note:**
Caution should be exercised when model spectra such as the Kaimal spectrum are used. In particular, it is important to beware that the true length scale may deviate significantly from the length scale $L_k$ of the model spectrum.

The Kaimal spectrum and other model spectra can be used to represent the upstream wind field in front of the wind turbine. However, a rotational sampling turbulence due to the rotation of the rotor blades will come in addition to the turbulence of the upstream wind field as represented by the model spectrum and will increase the wind fluctuations that the rotor blades effectively will experience.

For wind turbines located behind other wind turbines in a wind farm, the wind fluctuations represented by the model spectrum will become superimposed by an additional turbulence due to wake effects behind upstream wind turbines.

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### 3.2.4.4 The long-term probability distributions for the wind climate parameters $U_{10}$ and $\sigma_U$ that are interpreted from available data can be represented in terms of generic distributions or in terms of scattergrams. A typical generic distribution representation consists of a Weibull distribution for the 10-minute mean wind speed $U_{10}$ in conjunction with a lognormal distribution of $\sigma_U$ conditional on $U_{10}$. A scattergram provides the frequency of occurrence of given pairs $(U_{10}, \sigma_U)$ in a given discretisation of the $(U_{10}, \sigma_U)$ space.

### 3.2.4.5 Unless data indicate otherwise, a Weibull distribution can be assumed for the 10-minute mean wind speed $U_{10}$ in a given height $H$ above the seawater level,

$$F_{U_{10}}(u) = 1 - \exp\left(-\frac{u}{A}\right)^k$$

in which the scale parameter $A$ and the shape parameter $k$ are site- and height-dependent.

**Guidance note:**
In areas where hurricanes occur, the Weibull distribution as determined from available 10-minute wind speed records may not provide an adequate representation of the upper tail of the true distribution of $U_{10}$. In such areas, the upper tail of the distribution of $U_{10}$ needs to be determined on the basis of hurricane data.

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### 3.2.4.6 The wind speed profile represents the variation of the wind speed with height above the seawater level.

**Guidance note:**
A logarithmic wind speed profile may be assumed,

$$u(z) \propto \ln \frac{z}{z_0}$$

in which $z$ is the height above the seawater level and $z_0$ is a roughness parameter, which for offshore locations depends on the wind speed, the upstream distance to land, the water depth and the wave field. The logarithmic wind speed profile implies that the scale parameter $A(z)$ in height $z$ can be expressed in terms of the scale parameter $A(H)$ in...
The roughness parameter $z_0$ typically varies between 0.0001 m in open sea without waves and 0.003 m in coastal areas with onshore wind. The roughness parameter may be solved implicitly from the following equation:

$$z_0 = \frac{A_C}{\kappa} \left( \frac{k U_10}{g} \right)^{-\frac{1}{2}}$$

where $g$ is the acceleration of gravity, $\kappa = 0.4$ is von Karman’s constant, and $A_C$ is Charnock’s constant. For open sea with fully developed waves, $A_C = 0.011$ to 0.014 is recommended. For near-coastal locations, $A_C$ is usually higher with values of 0.018 or more. Whenever extrapolation of wind speeds to other heights than the height of the wind speed measurements is to be carried out, conservative worst-case values of $A_C$ should be applied.

As an alternative to the logarithmic wind profile, the power law profile may be assumed,

$$u(z) = U_{10}(H) \left( \frac{z}{H} \right)^{\alpha}$$

Offshore wind profiles can be governed more by atmospheric stability than by the roughness parameter $z_0$. For stability corrections of wind profiles reference is made to DNV-RP-C205.

3.2.4.7 Let $F_{U_{10}}(u)$ denote the long-term distribution of the 10-minute mean wind speed $U_{10}$. In areas where hurricanes do not occur, the distribution of the annual maximum 10-minute mean wind speed $U_{10,\text{max}}$ can be approximated by

$$F_{U_{10,\text{max,1 year}}}(u) = (F_{U_{10}}(u))^N$$

where $N = 52 560$ is the number of stationary 10-minute periods in one year.

**Guidance note:**

The quoted power-law approximation to the distribution of the annual maximum 10-minute mean wind speed is a good approximation to the upper tail of this distribution. Usually only quantiles in the upper tail of the distribution are of interest, viz. the 98% quantile which defines the 50-year mean wind speed. The upper tail of the distribution can be well approximated by a Gumbel distribution, whose expression is more operational than the quoted power-law expression.

Since the quoted power-law approximation to the distribution of the annual maximum 10-minute mean wind speed is used to estimate the 50-year mean wind speed by extrapolation, caution must be exercised when the underlying distribution $F_{U_{10}}$ of the arbitrary 10-minute mean wind speed is established. This applies in particular if $F_{U_{10}}$ is represented by the Weibull distribution of $U_{10}$ commonly used for prediction of the annual power production from the wind turbine, since this distribution is usually fitted to mid-range wind velocities and may not necessarily honour high-range wind speed data adequately.

In areas where hurricanes occur, the distribution of the annual maximum 10-minute mean wind speed $U_{10,\text{max}}$ shall be based on available hurricane data. This refers to hurricanes for which the 10-minute mean wind speed forms a sufficient representation of the wind climate.

3.2.4.8 The 10-minute mean wind speed with return period $T_R$ in units of years is defined as the $(1 - 1/T_R)$ quantile in the distribution of the annual maximum 10-minute mean wind speed, i.e. it is the 10-minute mean wind speed whose probability of exceedance in one year is $1/T_R$. It is denoted $U_{10,TR}$ and is expressed as

$$U_{10,T_R} = F_{U_{10,\text{max,1 year}}}^{-1}(1 - \frac{1}{T_R})$$

in which $T_R > 1$ year and $F_{U_{10,\text{max,1 year}}}$ denotes the cumulative distribution function of the annual maximum of the 10-minute mean wind speed.

The 10-minute mean wind speed with return period one year is defined as the mode of the cumulative distribution function of the annual maximum of the 10-minute mean wind speed.

**Guidance note:**

The 50-year 10-minute mean wind speed becomes $U_{10,50} = F_{U_{10,\text{max,1 year}}}^{-1}(0.98)$ and the 100-year 10-minute mean wind speed becomes $U_{10,100} = F_{U_{10,\text{max,1 year}}}^{-1}(0.99)$.  

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Note that these values, calculated as specified, are to be considered as central estimates of the respective 10-minute wind speeds when the underlying distribution function \( F_{U_{10,\text{max}}} \) is determined from limited data and is encumbered with statistical uncertainty.

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### 3.2.4.9 The natural variability of the wind speed

The natural variability of the wind speed about the mean wind speed \( U_{10} \) in a 10-minute period is known as turbulence and is characterised by the standard deviation \( \sigma_U \). For given values of \( U_{10} \), the standard deviation \( \sigma_U \) of the wind speed exhibits a natural variability from one 10-minute period to another. Caution should be exercised when fitting a distribution model to data for the standard deviation \( \sigma_U \). Often, the lognormal distribution provides a good fit to data for \( \sigma_U \) conditioned on \( U_{10} \), but use of a normal distribution, a Weibull distribution or a Frechet distribution is also seen. The choice of the distribution model may depend on the application, i.e., whether a good fit to data is required to the entire distribution or only in the body or the upper tail of the distribution.

**Guidance note:**
When the lognormal distribution is an adequate distribution model, the distribution of \( \sigma_U \) conditioned on \( U_{10} \) can be expressed as

\[
F_{\sigma_U | U_{10}} (\sigma) = \Phi\left( \frac{\ln \sigma - b_0}{b_1} \right)
\]

in which \( \Phi() \) denotes the standard Gaussian cumulative distribution function. The coefficients \( b_0 \) and \( b_1 \) are site-dependent coefficients dependent on \( U_{10} \).

The coefficient \( b_0 \) can be interpreted as the mean value of \( \ln \sigma_U \), and \( b_1 \) as the standard deviation of \( \ln \sigma_U \). The following relationships can be used to calculate the mean value \( E[\sigma_U] \) and the standard deviation \( D[\sigma_U] \) of \( \sigma_U \) from the values of \( b_0 \) and \( b_1 \),

\[
E[\sigma_U] = \exp(b_0 + \frac{1}{2} b_1^2)
\]

\[
D[\sigma_U] = E[\sigma_U] \sqrt{\exp(b_1^2) - 1}
\]

\( E[\sigma_U] \) and \( D[\sigma_U] \) will, in addition to their dependency on \( U_{10} \), also depend on local conditions, first of all the surface roughness \( z_0 \). Caution should be exercised when the distribution of \( \sigma_U \) conditioned on \( U_{10} \) is interpreted from data. It is important to identify and remove data, which belong to 10-minute series for which the stationarity assumption for \( U_{10} \) is not fulfilled. If this is not done, such data may confuse the determination of an appropriate distribution model for \( \sigma_U \) conditioned on \( U_{10} \). Techniques for “detrending” of data are available for application in the case that the mean wind speed follows a trend rather than stays stationary during a considered 10-minute period.

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### 3.2.4.10 Reference wind conditions and reference wind speeds

Let \( U_{10} \) and \( \sigma_U \) denote the 10-minute mean wind speed and the standard deviation of the wind speed, respectively, in a considered 10-minute period of stationary wind conditions. Unless data indicate otherwise, the short-term probability distribution for the instantaneous wind speed \( U \) at an arbitrary point in time during this period can be assumed to be a normal distribution. The cumulative distribution function for \( U \) can then be expressed as

\[
F_{U | \text{Normal} \sigma_U} (u) = \Phi\left( \frac{u - U_{10}}{\sigma_U} \right)
\]

in which \( \Phi() \) denotes the standard Gaussian cumulative distribution function.

**Guidance note:**
When data do not support the assumption of a normal distribution of the wind speed \( U \) conditioned on \( U_{10} \) and \( \sigma_U \), other generic distribution types may be tried out, and it may be necessary to introduce additional distribution parameters such as the skewness \( \alpha_U \) of the wind speed in order to arrive at an adequate representation of the wind speed distribution.

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### 3.2.4.11 Practical information regarding wind modelling

Practical information regarding wind modelling is given in DNV-RP-C205, in IEC61400-1 and in DNV/Risø Guidelines for Design of Wind Turbines.

### 3.2.5 Reference wind conditions and reference wind speeds

#### 3.2.5.1 For use in load combinations for design, a number of reference wind conditions and reference wind speeds are defined.

#### 3.2.5.2 The Normal Wind Profile (NWP) represents the average wind speed as a function of height above sea level.
Guidance note:
For standard wind turbine classes according to IEC61400-1, the normal wind profile is given by the power law model
with exponent $\alpha = 0.2$. For offshore locations it is recommended to apply an exponent $\alpha = 0.14$.

---end---of---Guidance---note---

3.2.5.3 The Normal Turbulence Model (NTM) represents turbulent wind speed in terms of a characteristic standard
deviation of wind speed, $\sigma_{U,c}$. The characteristic standard deviation $\sigma_{U,c}$ is defined as the 90% quantile
in the probability distribution of the standard deviation $\sigma_U$ of the wind speed conditioned on the 10-minute mean wind speed at the hub height.

For wind turbines within large wind farms, reference is in some cases made to the characteristic standard deviation
of the ambient wind speed, $\sigma_{Ua,c}$, where the ambient wind speed refers to the wind speed at the
individual wind turbine influenced by the presence of the other turbines. The characteristic standard deviation
of the ambient wind speed $\sigma_{Ua,c}$ is defined as the 90% quantile in the probability distribution of the standard deviation $\sigma_{Ua}$ of the ambient wind speed conditioned on the 10-minute mean wind speed at the hub height.

Guidance note:
For standard wind turbine classes according to IEC61400-1, prescribed values for the characteristic standard deviation $\sigma_{U,c}$ are given in IEC61400-1.

---end---of---Guidance---note---

3.2.5.4 The Extreme Wind Speed Model (EWM) is used to represent extreme wind conditions with a specified
return period, usually either one year or 50 years. It shall be either a steady wind model or a turbulent wind model. In case of a steady wind model, the extreme wind speed ($U_{EWM}$) at the hub height with a return period of 50 years shall be calculated as

$$U_{hub,50-yr} = 1.4 \cdot U_{10,hub,50-yr}$$

where $U_{10,hub,50-yr}$ denotes the 10-minute mean wind speed at the hub height with a return period of 50 years. The extreme wind speed ($U_{EWM}$) at the hub height with a return period of one year shall be calculated as

$$U_{hub,1-yr} = 0.8 \cdot U_{hub,50-yr}$$

The quantities $U_{hub,50-yr}$ and $U_{hub,1-yr}$ refer to wind speed averaged over three seconds. In the steady extreme wind model, allowance for short-term deviations from the mean wind direction shall be made by assuming constant yaw misalignment in the range of $\pm 15^\circ$.

The turbulent extreme wind model makes use of the 10-minute mean wind speed at the hub height with a return period of 50 years, $U_{10,hub,50-yr}$. The 10-minute mean wind speed at the hub height with a return period of one year shall be calculated as

$$U_{10,hub,1-yr} = 0.8 \cdot U_{10,hub,50-yr}$$

Further, for representation of turbulent wind speeds, the turbulent extreme wind model makes use of a
characteristic standard deviation of the wind speed. The characteristic standard deviation of the wind speed
shall be calculated as

$$\sigma_{U,c} = 0.11 \cdot U_{10,hub}$$

Guidance note:
For calculation of wind speeds and 10-minute mean wind speeds at other heights than the hub height, IEC61400-1 prescribes a wind profile given by the power law model with exponent $\alpha = 0.11$.

---end---of---Guidance---note---

3.2.5.5 The Extreme Operating Gust (EOG) at the hub height has a magnitude which shall be calculated as

$$V_{gust} = \min \left\{ 1.35(U_{hub,1-yr} - U_{10,hub}) + \frac{3.3 \sigma_{U,c}}{1 + 0.1D/A_i} \right\}$$

in which

$\sigma_{U,c}$ = characteristic standard deviation of wind speed, defined as the 90% quantile in the probability
distribution of $\sigma_U$. Inside large wind farms, the characteristic standard deviation $\sigma_{Ua,c}$ of the ambient
wind speed shall be used instead of $\sigma_{U,c}$. 

---end---of---Guidance---note---
\[ \Lambda_1 = \text{longitudinal turbulence scale parameter, is related to the integral scale parameter } L_k \text{ of the Kaimal spectral density through the relationship } L_k = 8.1 \Lambda_1. \]

\[ D = \text{rotor diameter.} \]

The wind speed \( V \) as a function of height \( z \) and time \( t \) shall be defined as follows

\[ V(z, t) = \begin{cases} u(z) - 0.37 \sqrt{\frac{2\pi t}{T}} \left( 1 - \cos \left( \frac{2\pi t}{T} \right) \right) \frac{u(z)}{u(z)} & \text{for } 0 \leq t \leq T \\ \theta_e & \text{otherwise} \end{cases} \]

where

\[ T = 10.5 \text{ sec} \]

and

\[ u(z) \] is defined by the Normal Wind Profile.

An example of extreme operating gust at the hub height is given in Figure 3-1 for a case where the 10-minute mean wind speed is 25 m/sec.

![Figure 3-1](image)

**Figure 3-1**  
**Example of extreme operating gust**

### 3.2.5.6 The Extreme Turbulence Model (ETM) combines the normal wind profile model NPM with a turbulent wind speed whose characteristic standard deviation is given by

\[ \sigma_{V,c} = c \cdot I_{ref} \cdot \left( 0.072 \cdot \frac{U_{\text{average}}}{c} + 3 \right) \left( \frac{U_{\text{hub}}}{c} - 4 \right) + 10 \]

in which

\[ c = 2 \text{ m/s} \]

\[ U_{\text{hub}} = \text{wind speed at hub height} \]

\[ U_{\text{average}} = \text{long-term average wind speed at hub height} \]

\[ I_{\text{ref}} = \text{expected value of turbulence intensity at hub height at } U_{10,\text{hub}} = 15 \text{ m/s} \]

### 3.2.5.7 The Extreme Direction Change (EDC) has a magnitude whose value shall be calculated according to the following expression

\[ \theta_e = \pm 4 \cdot \arctan \left( \frac{\sigma_{V,c}}{U_{10,\text{hub}}(1 + 0.1D/\Lambda_1)} \right) \]

where

\[ \sigma_{V,c} = \text{characteristic standard deviation of wind speed, defined according to the Normal Turbulence Model as the 90% quantile in the probability distribution of } \sigma_U. \text{ Inside large wind farms, the characteristic standard deviation } \sigma_{Ia,c} \text{ of the ambient wind speed shall be used instead of } \sigma_{V,c}. \]

\[ \Lambda_1 = \text{longitudinal turbulence scale parameter, is related to the integral scale parameter } L_k \text{ of the Kaimal spectral density through the relationship } L_k = 8.1 \Lambda_1. U_{10,\text{hub}} = 10\text{-minute mean wind speed at hub height} \]

\[ D = \text{rotor diameter.} \]

\( \theta_e \) is limited to the range \( \pm 180^\circ \).
The extreme direction change transient, $\theta(t)$, as a function of time $t$ shall be given by:

$$
\theta(t) = \begin{cases} 
0 & \text{for } t < 0 \\
0.5\theta_0(1 - \cos(\pi \cdot t / T)) & \text{for } 0 \leq t \leq T \\
0 & \text{for } t \geq T 
\end{cases}
$$

where $T = 6$ sec is the duration of the extreme direction change. The sign shall be chosen so that the worst transient loading occurs. At the end of the direction change transient, the direction is assumed to remain unchanged. The wind speed is assumed to follow the normal wind profile model given in [3.2.5.2].

As an example, the magnitude of the extreme direction change with a return period of one year is shown in Figure 3-2 for various values of $V_{\text{hub}} = U_{10,\text{hub}}$. The corresponding transient for $V_{\text{hub}} = U_{10,\text{hub}} = 25$ m/s is shown in Figure 3-3.

### Figure 3-2
Example of extreme direction change magnitude

### Figure 3-3
Example of extreme direction change

3.2.5.8 The Extreme Coherent Gust with Direction Change (ECD) shall have a magnitude of $V_{cg} = 15$ m/sec.

The wind speed $V$ as a function of height $z$ and time $t$ shall be defined as follows

$$
V(z,t) = \begin{cases} 
u(z) & \text{for } t < 0 \\
u(z) + 0.5V_{cg}(1 - \cos(\pi \cdot t / T)) & \text{for } 0 \leq t \leq T \\
u(z) + V_{cg} & \text{for } t > T 
\end{cases}
$$

where $T = 10$ sec is the rise time and $\nu(z)$ is the wind speed given in [3.2.5.2]. The extreme coherent gust is illustrated in Figure 3-4 for $V_{\text{hub}} = U_{10,\text{hub}} = 25$ m/s.
The rise in wind speed (described by the extreme coherent gust, see Figure 3-4) shall be assumed to occur simultaneously with the direction change $\theta$ from 0 degrees up to and including $\theta_{cg}$, where $\theta_{cg}$ is defined by:

$$\theta_{cg}(U_{10,\text{hub}}) = \begin{cases} 
180^\circ & \text{for } U_{10,\text{hub}} \leq 4 \text{ m/s} \\
\frac{720 \text{ m/s}}{U_{10,\text{hub}}} & \text{for } U_{10,\text{hub}} > 4 \text{ m/s} 
\end{cases}$$

The direction change $\theta_{cg}$ is shown in Figure 3-5 as a function of the 10-minute mean wind speed $V_{\text{hub}} = U_{10,\text{hub}}$ at hub height.

The direction change which takes place simultaneously as the wind speed rises is given by

$$\theta(t) = \begin{cases} 
0^\circ & \text{for } t < 0 \\
\pm 0.5\theta_{cg} (1 - \cos(\pi \cdot t / T)) & \text{for } 0 \leq t \leq T \\
\pm \theta_{cg} & \text{for } t > T 
\end{cases}$$

where $T = 10$ sec is the rise time. The normal wind profile model as specified in [3.2.5.2] shall be used. An example of the direction change is shown in Figure 3-6 as a function of time for $V_{\text{hub}} = U_{10,\text{hub}} = 25$ m/s.
3.2.5.9 The Extreme Wind Shear model (EWS) is used to account for extreme transient wind shear events. It consists of a transient vertical wind shear and a transient horizontal wind shear. The extreme transient positive and negative vertical shear shall be calculated as

$$V(z,t) = \left\{ \begin{array}{ll} \frac{U_{10}(z) \pm \frac{z - z_{hub}}{D}}{U_{10}(z)} \left[ 2.5 + 0.2 \beta_{\sigma_c} \left( \frac{D}{z} \right) \right] \left( 1 - \cos\frac{2 \pi t}{T} \right) & \text{for } 0 \leq t \leq T \\ \text{otherwise} \end{array} \right. $$

The extreme transient horizontal shear shall be calculated as

$$V(y,t) = \left\{ \begin{array}{ll} \frac{U_{10}(z) \pm \frac{y}{D}}{U_{10}(z)} \left[ 2.5 + 0.2 \beta_{\sigma_c} \left( \frac{D}{z} \right) \right] \left( 1 - \cos\frac{2 \pi t}{T} \right) & \text{for } 0 \leq t \leq T \\ \text{otherwise} \end{array} \right. $$

Here, $U_{10}(z)$ denotes the wind shear profile according to the Normal Wind Profile model, $z$ is the height above sea level, $z_{hub}$ is the hub height, $y$ is the lateral cross-wind distance, $\Lambda_1$ is the longitudinal turbulence scale parameter, $\sigma_{U,c}$ is the characteristic standard deviation of wind speed, defined according to the Normal Turbulence Model as the 90% quantile in the probability distribution of $\sigma_U$, $D$ is the rotor diameter, $\beta = 6.4$ and $T = 12$ sec. Inside large wind farms, the characteristic standard deviation $\sigma_{Ua,c}$ of the ambient wind speed shall be used instead of $\sigma_{U,c}$.

The sign for the horizontal wind shear transient shall be chosen in such a manner that the most unfavourable transient loading occurs. The extreme transient horizontal shear and the extreme transient vertical shear shall not be applied simultaneously.

**Guidance note:**
For standard wind turbine classes according to IEC61400-1, the normal wind profile $U_{10}(z)$ is given by the power law model with exponent $\alpha = 0.2$. For offshore locations it is recommended to apply an exponent $\alpha = 0.14$.

3.2.5.10 The Reduced Wind Speed Model (RWM) defines a companion wind speed $U_{RWM}$ to be used in combination with the extreme wave height (EWH) for definition of an extreme event with a specified return period. The reduced wind speed can be expressed as a fraction of the extreme wind speed, $U_{RWM} = \psi \cdot U_{EWM}$, $\psi < 1$. The Reduced Wind Speed is used for definition of events with return periods of 50 years and 1 year, and the corresponding reduced wind speeds are denoted $U_{R,50-yr}$ and $U_{R,1-yr}$, respectively.

**Guidance note:**
IEC61400-3 requires use of $U_{R,50-yr} = 1.1 U_{10,50-yr}$, which implies $\psi = 0.79$. Other values for $\psi$ can be applied, provided they can be substantiated by site-specific data.

3.2.5.11 Caution should be exercised when using site-specific wind measurements for the calculation of the extreme wind conditions EOG, EDC, ECD and EWS. The extreme wind conditions are to be representative values with return periods of 50 years. Site-specific measured wind speeds may not be of a sufficiently long period to capture extreme trends properly. Use of site-specific measured wind speeds for the calculation of the extreme wind conditions may therefore be insufficient. In such cases, use of the basic parameters for the IEC Wind Turbine Classes given in IEC61400-1 may be considered.
3.3 Wave climate

3.3.1 Wave parameters

3.3.1.1 The wave climate is represented by the significant wave height $H_S$ and the spectral peak period $T_P$. In the short term, i.e. over a 3-hour or 6-hour period, stationary wave conditions with constant $H_S$ and constant $T_P$ are assumed to prevail.

**Guidance note:**

The significant wave height $H_S$ is defined as four times the standard deviation of the sea elevation process. The significant wave height is a measure of the intensity of the wave climate as well as of the variability in the arbitrary wave heights. The peak period $T_P$ is related to the mean zero-crossing period $T_Z$ of the sea elevation process.

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

3.3.1.2 The wave height $H$ of a wave cycle is the difference between the highest crest and the deepest trough between two successive zero-upcrossings of the sea elevation process. The arbitrary wave height $H$ under stationary 3- or 6-hour conditions in the short term follows a probability distribution which is a function of the significant wave height $H_S$.

3.3.1.3 The wave period is defined as the time between two successive zero-upcrossings of the sea elevation process. The arbitrary wave period $T$ under stationary 3- or 6-hour conditions in the short term follows a probability distribution, which is a function of $H_S$, $T_P$ and $H$.

3.3.1.4 The wave crest height $H_C$ is the height of the highest crest between two successive zero-upcrossings of the sea elevation process. The arbitrary wave crest height $H_C$ under stationary 3- or 6-hour conditions in the short term follows a probability distribution which is a function of the significant wave height $H_S$.

3.3.1.5 The short term 3- or 6-hour sea state may be represented by a wave spectrum, i.e. the power spectral density function of the sea elevation process, $S(f)$. $S(f)$ is a function of $H_S$ and $T_P$ and expresses how the energy of the sea elevation is distributed between various frequencies.

3.3.2 Wave data

3.3.2.1 Wave statistics are to be used as a basis for representation of the long-term and short-term wave conditions. Empirical statistical data used as a basis for design must cover a sufficiently long period of time, preferably 10 years or more.

**Guidance note:**

Wave data obtained on site are to be preferred over wave data observed at an adjacent location. Good quality data are measured data and hindcast data. Continuous records of data are to be preferred over records with gaps. Longer periods of observation are to be preferred over shorter periods.

When no site-specific wave data are available and data from adjacent locations are to be capitalised on in stead, proper transformation of such other data shall be performed to account for possible differences due to different water depths and different seabed topographies. Such transformation shall take effects of shoaling and refraction into account.

Hindcast of wave data may be used to extend measured time series, or to interpolate to places where measured data have not been collected. If hindcast is used, the hindcast model shall be calibrated against measured data to ensure that the hindcast results comply with available measured data.

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

3.3.2.2 The long-term distributions of $H_S$ and $T_P$ should preferably be based on statistical data for the same reference period for the waves as the reference period which is used for the determination of loads. If a different reference period than 3 or 6 hours is used for the determination of loads, the wave data may be converted by application of appropriate adjustment factors.

**Guidance note:**

When the long-term distribution of the arbitrary significant wave height $H_S$ is given by a Weibull distribution,

$$F_{H_S}(h) = 1 - \exp\left(-\frac{h}{h_0}\right)^\beta$$

the significant wave height $H_{S,T_S}$ for a reference period of duration $T_S$ can be obtained from the significant wave height $H_{S,T_{S_0}}$ for a reference period of duration $T_{S_0}$ according to the following relationship,

$$H_{S,T_S} = H_{S,T_{S_0}} \left[1 + \frac{\ln(T_{S_0}/T_S)}{\ln(N_T R)}\right]^{\frac{1}{\beta}}$$
in which \( N_0 \) is the number of sea states of duration \( T_{S0} \) in one year and \( T_R \) is the specified return period of the significant wave height, which is to be converted. \( N_0 = 2920 \) when \( T_{S0} = 3 \) hours. \( T_R \) must be given in units of years.

---end of Guidance note---

3.3.2.3 Wave climate and wind climate are correlated, because waves are usually wind-generated. The correlation between wave data and wind data shall be accounted for in design.

Guidance note:
Simultaneous observations of wave and wind data in terms of simultaneous values of \( H_S \) and \( U_{10} \) should be obtained. It is recommended that directionality of wind and waves are recorded. Extreme waves may not always come from the same direction as extreme winds. This may in particular be so when the fetch in the direction of the extreme winds is short.

Within a period of stationary wind and wave climates, individual wind speeds and wave heights can be assumed independent and uncorrelated.

---end of Guidance note---

3.3.3 Wave modelling

3.3.3.1 Site-specific spectral densities of the sea elevation process can be determined from available wave data. When selecting a model for representation of the spectral density of the sea elevation process, it is important to consider both wind sea and swell.

3.3.3.2 Unless data indicate otherwise, the spectral density of the sea elevation process may be represented by the JONSWAP spectrum,

\[
S(f) = \frac{\alpha g^2}{(2\pi)^4} f^{-5} \exp\left(-\frac{5}{4} \left(\frac{f}{f_p}\right)^4\right) \exp\left(-0.5 \left(\frac{f-f_p}{\sigma f_p}\right)^2\right)
\]

where

\( f = \) wave frequency, \( f = 1/T \)
\( T = \) wave period
\( f_p = \) spectral peak frequency, \( f_p = 1/T_p \)
\( T_p = \) peak period
\( g = \) acceleration of gravity
\( \alpha = \) generalised Phillips’ constant
\( = 5 \cdot (H_S^{2} f_p^4/g^2) \cdot (1-0.287 \ln \gamma) \cdot \pi^4 \)
\( \sigma = \) spectral width parameter
\( = 0.07 \) for \( f \leq f_p \) and \( \sigma = 0.09 \) for \( f > f_p \)
\( \gamma = \) peak-enhancement factor.

The zero-upcrossing period \( T_Z \) depends on the peak period \( T_p \) through the following relationship,

\[
T_Z = T_p \frac{5 + \gamma}{11 + \gamma}
\]

The peak-enhancement factor is

\[
\gamma = \begin{cases} 
5 & \text{for } \frac{T_p}{\sqrt{H_S}} \leq 3.6 \\
\exp(5.75-1.15 \frac{T_p}{\sqrt{H_S}}) & \text{for } 3.6 < \frac{T_p}{\sqrt{H_S}} \leq 5 \\
1 & \text{for } 5 < \frac{T_p}{\sqrt{H_S}}
\end{cases}
\]

where \( T_p \) is in seconds and \( H_S \) is in metres.

The JONSWAP spectrum may not necessarily suffice for representation of a sea elevation process with a significant swell component. When the sea elevation process has a significant swell component, a two-peaked spectrum such as the Torsethaugen spectrum may form a better representation of the spectral density of this process than the JONSWAP spectrum. Details of the Torsethaugen spectrum are given in DNV-RP-C205.
When $\gamma = 1$ the JONSWAP spectrum reduces to the Pierson-Moskowitz spectrum.

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

3.3.3.3 The long-term probability distributions for the wave climate parameters $H_S$ and $T_P$ that are interpreted from available data can be represented in terms of generic distributions or in terms of scattergrams. A typical generic distribution representation consists of a Weibull distribution for the significant wave height $H_S$ in conjunction with a lognormal distribution of $T_P$ conditional on $H_S$. A scattergram gives the frequency of occurrence of given pairs $(H_S, T_P)$ in a given discretisation of the $(H_S, T_P)$ space.

3.3.3.4 Unless data indicate otherwise, a 3-parameter Weibull distribution can be assumed for the significant wave height,

$$F_{H_S}(h) = 1 - \exp\left(-\left(\frac{h - \gamma}{\alpha}\right)^\beta\right)$$

3.3.3.5 When $F_{H_S}(h)$ denotes the distribution of the significant wave height in an arbitrary t-hour sea state, the distribution of the annual maximum significant wave height $H_{S,\text{max}}$ can be taken as

$$F_{H_{S,\text{max},1\text{year}}}(h) = (F_{H_S}(h))^N$$

where $N$ is the number of t-hour sea states in one year. For $t = 3$ hours, $N = 2920$.

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

3.3.3.6 The significant wave height with return period $T_R$ in units of years is defined as the $(1 - 1/T_R)$ quantile in the distribution of the annual maximum significant wave height, i.e. it is the significant wave height whose probability of exceedance in one year is $1/T_R$. It is denoted $H_{S,T_R}$ and is expressed as

$$H_{S,T_R} = F_{H_{S,\text{max},1\text{year}}}^{-1}\left(1 - \frac{1}{T_R}\right)$$

in which $T_R > 1$ year.

The significant wave height with return period one year is defined as the mode of the distribution function of the annual maximum of the significant wave height.

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

3.3.3.7 In deep waters, the short-term probability distribution of the arbitrary wave height $H$ can be assumed to follow a Rayleigh distribution when the significant wave height $H_S$ is given,

$$F_{H|H_S}(h) = 1 - \exp\left(-\frac{2h^2}{(1-\nu^2)H_S^2}\right)$$

where $F_{H|H_S}$ denotes the cumulative distribution function and $\nu$ is a spectral width parameter whose value is $\nu = 0.0$ for a narrow-banded sea elevation process.

The maximum wave height $H_{\text{max}}$ in a 3-hour sea state characterised by a significant wave height $H_S$ can be calculated as a constant factor times $H_S$. 

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---
Guidance note:
The maximum wave height in a sea state can be estimated by the mean of the highest wave height in the record of waves that occur during the sea state, or by the most probable highest wave height in the record. The most probable highest wave height is also known as the mode of the highest wave height. Both of these estimates for the maximum wave height in a sea state depend on the number of waves, \( N \), in the record. \( N \) can be defined as the ratio between the duration \( T_S \) of the sea state and the mean zero-upcrossing period \( T_Z \) of the waves. For a narrow-banded sea elevation process, the appropriate expression for the mean of the highest wave height \( H_{\text{max,mean}} \) reads

\[
H_{\text{max,mean}} = \left[ \frac{1}{2} \ln N + \frac{0.2886}{\sqrt{2 \ln N}} \right] H_S
\]

while the expression for the mode of the highest wave height reads

\[
H_{\text{max,mode}} = \left[ \frac{1}{2} \ln N \right] H_S
\]

For a sea state of duration \( T_S = 3 \) hours and a mean zero-upcrossing period \( T_Z \) of about 10.8 sec, \( N = 1000 \) results. For this example, the mean of the highest wave height becomes \( H_{\text{max}} = 1.936 H_S \approx 1.94 H_S \), while the mode of the highest wave height becomes \( H_{\text{max}} = 1.858 H_S \approx 1.86 H_S \). For shorter mean zero-upcrossing periods than the assumed 10.8 sec, \( N \) becomes larger, and so does the factor on \( H_S \).

Table 3-1 gives the ratio \( H_{\text{max}}/H_S \) for various values of \( N \).

<table>
<thead>
<tr>
<th>No. of waves ( N = T_S/T_Z )</th>
<th>Ratio ( H_{\text{max}}/H_S ) mode</th>
<th>Ratio ( H_{\text{max}}/H_S ) mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>500</td>
<td>1.763</td>
<td>1.845</td>
</tr>
<tr>
<td>1000</td>
<td>1.858</td>
<td>1.936</td>
</tr>
<tr>
<td>1500</td>
<td>1.912</td>
<td>1.988</td>
</tr>
<tr>
<td>2000</td>
<td>1.949</td>
<td>2.023</td>
</tr>
<tr>
<td>2500</td>
<td>1.978</td>
<td>2.051</td>
</tr>
<tr>
<td>5000</td>
<td>2.064</td>
<td>2.134</td>
</tr>
</tbody>
</table>

Other ratios than those quoted in Table 3-1 apply to waves in shallow waters and in cases where the sea elevation process is not narrow-banded.

It is common to base the estimation of \( H_{\text{max}} \) on the results for the mode rather than on the results for the mean. Table 3-1 is valid for \( H_S/d < 0.2 \), where \( d \) denotes water depth.

3.3.3.8 In shallow waters, the wave heights will be limited by the water depth. Unless data indicate otherwise, the maximum wave height can be taken as 78% of the water depth. The Rayleigh distribution of the wave heights will become distorted in the upper tail to approach this limit asymptotically. Use of the unmodified Rayleigh distribution for representation of the distribution of wave heights in shallow waters may therefore be on the conservative side.

3.3.3.9 In shallow waters with constant seabed slope, the Battjes and Groenendijk distribution can be used to represent the probability distribution of the arbitrary wave height \( H \) conditional on the significant wave height \( H_S \). It is a requirement for this use of the Battjes and Groenendijk distribution that it is validated by measured site-specific wave data. The Battjes and Groenendijk distribution is a composite Weibull distribution whose cumulative distribution function reads

\[
F_{w_{\text{BG}}} (h) = \begin{cases} 
1 - \exp\left(-\left(\frac{h}{h_1}\right)^{\alpha}\right) & \text{for } h \leq h_T \\
1 - \exp\left(-\left(\frac{h}{h_2}\right)^{\alpha}\right) & \text{for } h > h_T 
\end{cases}
\]

in which the transition wave height \( h_T \) is defined as

\[
h_T = (0.35 + 5.8 \cdot \tan \alpha) \cdot d
\]

where \( \alpha \) is the slope angle of the sea floor and \( d \) is the water depth. The parameters \( h_1 \) and \( h_2 \) are functions of the transition wave height \( h_T \) and of the root mean square \( H_{\text{RMS}} \) of the wave heights. The root mean square...
$H_{\text{RMS}}$ is calculated from the significant wave height $H_S$ and the water depth $d$ as

$$h_{\text{rms}} = 0.6725H_s + 0.2025 \frac{H_{\text{RMS}}^2}{d}$$

and the parameters $h_1$ and $h_2$ can be found from the following approximate expressions, valid for $0.05H_{\text{RMS}} < h_T < 3H_{\text{RMS}}$:

$$\frac{h_t}{H_{\text{RMS}}} = 0.0835 \left( \frac{h_t}{H_{\text{RMS}}} \right)^3 - 0.583 \left( \frac{h_t}{H_{\text{RMS}}} \right)^2 + 1.3339 \left( \frac{h_t}{H_{\text{RMS}}} \right)$$

$$\frac{h_t}{H_{\text{RMS}}} = 1.06 - 0.01532 \left( \frac{h_t}{H_{\text{RMS}}} \right)^2$$

$$+ 0.083259 \left( \frac{h_t}{H_{\text{RMS}}} \right)^3 - 0.01925 \left( \frac{h_t}{H_{\text{RMS}}} \right)^4$$

The Battjes and Groenendijk distribution is not defined for $h_T > 3H_{\text{RMS}}$.

**Guidance note:**

The Battjes and Groenendijk distribution has the drawback that it has an unphysical “knee” at the transition height $h_T$. Its upper-tail behaviour may also be of concern. The Battjes and Groenendijk distribution should therefore be used with caution and only when supported by data.

Other distribution models for wave heights in shallow waters exist and can be used as alternatives to the Battjes and Groenendijk distribution as long as they provide an adequate representation of the true distribution of the wave heights. Examples of such distribution models include the Glukowski, Bitner and Næss distribution models.

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

3.3.3.10 The long-term probability distribution of the arbitrary wave height $H$ can be found by integration over all significant wave heights

$$F_{H}(h) = \frac{1}{N_W} \int \int \nu_0(h_s,t) \cdot F_{H|H_sT_P}(h_s) \cdot f_{H|H_sT_P}(h_s,t) dh_s$$

where

$$F_0 = \int \int \nu_0(h_s,t) \cdot f_{H|H_sT_P}(h_s,t) dh_s$$

in which $f_{H|H_sT_P}(h_s,t)$ is the joint probability density of the significant wave height $H_s$ and the peak period $T_P$ and $\nu_0(h_s,t)$ is the zero-upcrossing rate of the sea elevation process for given combination of $H_s$ and $T_P$. $F_{H|H_sT_P}(h)$ denotes the short-term cumulative distribution function for the wave height $H$ conditioned on $H_s$ and $T_P$.

3.3.3.11 When $F_{H}(h)$ denotes the distribution of the arbitrary wave height $H$, the distribution of the annual maximum wave height $H_{\text{max}}$ can be taken as

$$F_{H_{\text{max}},1\text{ year}}(h) = (F_{H}(h))^{N_W}$$

where $N_W$ is the number of wave heights in one year.

3.3.3.12 Unless data indicate otherwise, the wave crest height $H_C$ can be assumed to be 0.65 times the associated arbitrary wave height $H$.

3.3.3.13 The wave height with return period $T_R$ in units of years is defined as the $(1 - 1/T_R)$ quantile in the distribution of the annual maximum wave height, i.e. it is the wave height whose probability of exceedance in one year is $1/T_R$. It is denoted $H_{T_R}$ and is expressed as

$$H_{T_R} = F_{H_{\text{max}},1\text{ year}}^{-1}(1 - \frac{1}{T_R})$$

in which $T_R > 1$ year.

The wave height with return period one year is defined as the mode of the distribution function of the annual maximum wave height.

**Guidance note:**

The 50-year wave height becomes $H_{50} = F_{H_{\text{max}},1\text{ year}}^{-1}(0.98)$ and the 100-year wave height becomes $H_{100} = F_{H_{\text{max}},1\text{ year}}^{-1}(0.99)$. Note that these values, calculated as specified, are to be considered as central estimates of the respective...
wave heights when the underlying distribution function \( F_{\text{hmax}} \) is determined from limited data and is encumbered with statistical uncertainty.

Note also that the 50-year wave height \( H_{50} \) is always greater than the maximum wave height \( H_{\text{max}} \) in the 3-hour sea state whose return period is 50 years and whose significant wave height is denoted \( H_{S,50} \). This implies that in deep waters \( H_{50} \) will take on a value greater than \( H_{\text{max}} = 1.86H_{S,50} \). Values of \( H_{50} \) equal to about 2.0 times \( H_{S,50} \) are not uncommon in deep waters.

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

3.3.3.14 The design wave crest shall be taken as the wave crest height which has a return period of 50 years.

3.3.3.15 Directionality of waves shall be considered for determination of wave height distributions and wave heights with specified return periods when such directionality has an impact on the design of a wind turbine structure.

3.3.4 Reference sea states and reference wave heights

3.3.4.1 For use in load combinations for design, a number of reference sea states and reference wave heights are defined.

3.3.4.2 The Normal Sea State (NSS) is characterised by a significant wave height, a peak period and a wave direction. It is associated with a concurrent mean wind speed. The significant wave height \( H_{S,NSS} \) of the normal sea state is defined as the expected value of the significant wave height conditioned on the concurrent 10-minute mean wind speed. The normal sea state is used for calculation of ultimate loads and fatigue loads. For fatigue load calculations a series of normal sea states have to be considered, associated with different mean wind speeds. It must be ensured that the number and resolution of these normal sea states are sufficient to predict the fatigue damage associated with the full long-term distribution of metocean parameters. The range of peak periods \( T_p \) appropriate to each significant wave height shall be considered. Design calculations shall be based on values of the peak period which result in the highest loads or load effects in the structure.

3.3.4.3 The Normal Wave Height (NWH) \( H_{NWH} \) is defined as the expected value of the significant wave height conditioned on the concurrent 10-minute mean wind speed, i.e. \( H_{NWH} = H_{S,NSS} \). The range of wave periods \( T \) appropriate to the normal wave height shall be considered. Design calculations shall be based on values of the wave period within this range that result in the highest loads or load effects in the structure.

Guidance note:

In deep waters, the wave periods \( T \) to be used with \( H_{NWH} \) may be assumed to be within the range given by

\[
11.1 \frac{H_{S,NSS}(U_{10})}{g} \leq T \leq 14.3 \sqrt{H_{S,NSS}(U_{10}) g}
\]

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

3.3.4.4 The Severe Sea State (SSS) is characterised by a significant wave height, a peak period and a wave direction. It is associated with a concurrent mean wind speed. The significant wave height of the severe sea state \( H_{S,SSS} \) is defined by extrapolation of appropriate site-specific metocean data such that the load effect from the combination of the significant wave height \( H_{S,SSS} \) and the 10-minute mean wind speed \( U_{10} \) has a return period of 50 years. The SSS model is used in combination with normal wind conditions for calculation of the ultimate loading of an offshore wind turbine during power production. The SSS model is used to associate a severe sea state with each mean wind speed in the range corresponding to power production. For all 10-minute mean wind speeds \( U_{10} \) during power production, the unconditional extreme significant wave height, \( H_{S,SSS}(U_{10}) \), with a return period of 50 years may be used as a conservative estimate for \( H_{S,SSS}(U_{10}) \). Further guidance regarding estimation of \( H_{S,SSS} \) is provided in [4.6.7.3]. The range of peak periods \( T_p \) appropriate to each significant wave height shall be considered. Design calculations shall be based on values of the peak period which result in the highest loads or load effects in the structure.

3.3.4.5 The Severe Wave Height (SWH) \( H_{SWH} \) is associated with a concurrent mean wind speed and is defined by extrapolation of appropriate site-specific metocean data such that the load effect from the combination of the severe wave height \( H_{SWH} \) and the 10-minute mean wind speed \( U_{10} \) has a return period of 50 years. The SWH model is used in combination with normal wind conditions for calculation of the ultimate loading of an offshore wind turbine during power production. The SWH model is used to associate a severe wave height with each mean wind speed in the range corresponding to power production. For all 10-minute mean wind speeds \( U_{10} \) during power production, the unconditional extreme wave height, \( H_{50-yr} \), with a return period of 50 years may be used as a conservative estimate for \( H_{SWH}(U_{10}) \). The range of wave periods \( T_p \) appropriate to the severe wave height shall be considered. Design calculations shall be based on values of the wave period within this range that result in the highest loads or load effects in the structure.
Guidance note:
In deep waters, the wave periods $T$ to be used with $H_{SWH}$ may be assumed to be within the range given by

$$11.1 \sqrt{H_{S,ESS}(U_o) / g} \leq T \leq 14.3 \sqrt{H_{S,ESS}(U_o) / g}$$

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

3.3.4.6 The Extreme Sea State (ESS) is characterised by a significant wave height, a peak period and a wave direction. The significant wave height $H_{S,ESS}$ is the unconditional significant wave height with a specified return period, determined from the distribution of the annual maximum significant wave height as outlined in [3.3.3.6]. The Extreme Sea State is used for return periods of 50 years and 1 year, and the corresponding significant wave heights are denoted $H_{S,50-yr}$ and $H_{S,1-yr}$, respectively. The range of peak periods $T_p$ appropriate to each of these significant wave heights shall be considered. Design calculations shall be based on values of the peak period which result in the highest loads or load effects in the structure.

3.3.4.7 The Extreme Wave Height (EWH) $H_{EWH}$ is a wave height with a specified return period. It can be determined from the distribution of the annual maximum wave height as outlined in [3.3.3.13]. In deep waters, it can be estimated based on the significant wave height $H_{S,ESS}$ with the relevant return period as outlined in [3.3.3.7]. The Extreme Wave Height is used for return periods of 50 years and 1 year, and the corresponding wave heights are denoted $H_{EWH,50-yr}$ and $H_{EWH,1-yr}$, respectively. The range of wave periods $T$ appropriate to the severe wave height shall be considered. Design calculations shall be based on values of the wave period within this range that result in the highest loads or load effects in the structure.

Guidance note:
In deep waters, the wave periods $T$ to be used with $H_{EWH}$ may be assumed to be within the range given by

$$11.1 \sqrt{H_{S,ESS}(U_o) / g} \leq T \leq 14.3 \sqrt{H_{S,ESS}(U_o) / g}$$

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

3.3.4.8 The Reduced Wave Height (RWH) $H_{RWH}$ is a companion wave height to be used in combination with the extreme wind speed (EWS) for definition of an extreme event with a specified return period. The reduced wave height can be expressed as a fraction of the extreme wave height, $H_{RWH} = \psi \cdot H_{EWH}$, $\psi < 1$. The Reduced Wave Height is used for definition of events with return periods of 50 years and 1 year, and the corresponding reduced wave heights are denoted $H_{RWH,50-yr}$ and $H_{RWH,1-yr}$, respectively.

Guidance note:
It is practice for offshore structures to apply $\psi = H_{5-yr}/H_{50-yr}$, where $H_{5-yr}$ and $H_{50-yr}$ denote the individual wave heights with 5- and 50-year return period, respectively. The shallower the water depth, the larger is usually the value of $\psi$.

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

3.3.4.9 The wave period $T$ associated with the wave heights in [3.3.3.3], [3.3.4.5], [3.3.4.7] and [3.3.4.8] has a depth-dependent lower limit derived from wave breaking considerations,

$$T > \sqrt{\frac{34.5 d}{g} \tanh^{-1} \left( \frac{H}{0.78d} \right)}$$

where $H$ is the wave height, $d$ is the water depth and $g$ is the acceleration of gravity.

3.3.5 Wave theories and wave kinematics

3.3.5.1 The kinematics of regular waves may be represented by analytical or numerical wave theories, which are listed below:

- linear wave theory (Airy theory) for small-amplitude deep water waves; by this theory the wave profile is represented by a sine function
- Stokes wave theories for high waves
- stream function theory, based on numerical methods and accurately representing the wave kinematics over a broad range of water depths
- Boussinesq higher-order theory for shallow water waves
- solitary wave theory for waves in particularly shallow water.

3.3.5.2 Three wave parameters determine which wave theory to apply in a specific problem. These are the wave height $H$, the wave period $T$ and the water depth $d$. These parameters are used to define three non-dimensional parameters that determine ranges of validity of different wave theories,

- Wave steepness parameter: $S = 2\pi \frac{H}{gT^2} = \frac{H}{\lambda_o}$
— Shallow water parameter: \[ \mu = 2\pi \frac{d}{gT^2} = \frac{d}{\lambda_0} \]

— Ursell parameter: \[ U'_r = \frac{H}{k'_0 d'} = \frac{1}{4\pi} \frac{S}{\mu'} \]

where \( \lambda_0 \) and \( \kappa_0 \) are the linear deepwater wavelength and wave number corresponding to wave period \( T \). The ranges of application of the different wave theories are given in Table 3-2.

<table>
<thead>
<tr>
<th>Theory</th>
<th>Application</th>
<th>Approximate range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear (Airy) wave</td>
<td>Deep and shallow</td>
<td>( S &lt; 0.006; S/\mu &lt; 0.03 )</td>
</tr>
<tr>
<td>2\text{nd} order Stokes wave</td>
<td>Deep water</td>
<td>( U_r &lt; 0.65; S &lt; 0.04 )</td>
</tr>
<tr>
<td>5\text{th} order Stokes wave</td>
<td>Deep water</td>
<td>( U_r &lt; 0.65; S &lt; 0.14 )</td>
</tr>
<tr>
<td>Cnoidal theory</td>
<td>Shallow water</td>
<td>( U_r &gt; 0.65; \mu &lt; 0.125 )</td>
</tr>
</tbody>
</table>

Figure 3-7 shows the ranges of validity for different wave theories.

3.3.5.3 Linear wave theory is the simplest wave theory and is obtained by taking the wave height to be much smaller than both the wavelength and the water depth, or equivalently

\[ S \ll 1 \quad ; \quad U_r \ll 1 \]

This theory is referred to as small amplitude wave theory, linear wave theory, sinusoidal wave theory or as Airy theory.

For regular linear waves the wave crest height \( A_C \) is equal to the wave trough height \( A_H \) and denoted the wave amplitude \( A \), hence \( H = 2A \).

The surface elevation is given by

\[ \eta(x,y,t) = \frac{H}{2} \cos \Theta \]

where \( \Theta = k(x \cos \beta + y \sin \beta - ct) \) and \( \beta \) is the direction of propagation, measured from the positive x-axis.
The dispersion relationship gives the relationship between wave period \( T \) and wavelength \( \lambda \). For waves in waters with finite water depth \( d \) the dispersion relationship is given by the transcendental equation

\[
\lambda = \frac{g T^2}{2\pi} \tanh \left( \frac{2\pi d}{\lambda} \right)
\]

in which \( g \) denotes the acceleration of gravity. A good approximation to the wavelength \( \lambda \) as a function of the wave period \( T \) is given by

\[
\lambda = T (gd)^{1/2} \left( \frac{f(\sigma)}{1 + \sigma f(\sigma)} \right)^{1/2}
\]

where

\[
f(\sigma) = 1 + \sum_{n=1}^{4} \alpha_n \sigma^n
\]

and

\[
\sigma = \left( \frac{4\pi^2 d}{g T^2} \right)
\]

\( \alpha_1 = 0.666, \alpha_2 = 0.445, \alpha_3 = -0.105, \alpha_4 = 0.272. \)

3.3.5.4 Stokes wave theory implies the Stokes wave expansion, which is an expansion of the surface elevation in powers of the linear wave height \( H \). A Stokes wave expansion can be shown to be formally valid for

\( S < \frac{1}{10} \); \( U_r < 1 \)

A first-order Stokes wave is identical to a linear wave, or Airy wave. A second-order Stokes wave is a reasonably accurate approximation when

\( S < 0.04 \) and \( U_r < 0.65 \)

The surface elevation profile for a regular second-order Stokes wave is given by

\[
\eta = \frac{H}{2} \cos \Theta + \frac{2H^2}{8\alpha} \cosh kd \left[ 2 + \cosh 2kd \right] \cos 2\Theta
\]

where \( \Theta = k (x \cos \beta + y \sin \beta - ct) \). Second-order and higher order Stokes waves are asymmetric with \( A_C > A_T \). Crests are steeper and troughs are wider than for Airy waves.

The linear dispersion relation holds for second-order Stokes waves, hence the phase velocity \( c \) and the wavelength \( \lambda \) remain independent of wave height.

To third order, the phase velocity depends on wave height according to

\[
c^2 = \frac{g}{k} \tanh(kd) \left\{ 1 + \left( \frac{kH}{2} \right)^2 \left[ 9 - 8 \cosh^2(kd) + 8 \cosh^4(kd) \right] \right\}
\]

The wave height is limited by breaking. The maximum steepness is

\( S_{\text{max}} = \frac{H}{\lambda} = 0.142 \tanh \frac{2\pi d}{\lambda} \)

where \( \lambda \) is the wavelength corresponding to water depth \( d \).

For deep water the breaking wave limit is approximated by \( S_{\text{max}} = 1/7 \).

Use of second order Stokes waves is limited by the steepness criterion

\[
kH = 0.924 \sinh \left( \frac{kd}{\sqrt{1 + 8 \cosh^2 kd}} \right)
\]

For regular steep waves \( S < S_{\text{max}} \) (and \( U_r < 0.65 \)) Stokes fifth order wave theory applies. Stokes wave theory is not applicable for very shallow waters.

3.3.5.5 Cnoidal wave theory defines a wave which is a periodic wave with sharp crests separated by wide troughs. The range of validity of cnoidal wave theory is

\( \mu < 0.125 \) and \( U_r > 0.65 \)

The surface profile of cnoidal waves of wave height \( H \) and period \( T \) in water depth \( d \) is given by

\[
\eta(x,t) = \frac{16d^2}{3\lambda^2} \left\{ K(k) \left[ K(k) - E(k) \right] \right\} + 1 - \frac{H}{d}
\]

\[
+ Hcn^2 \left[ 2K(k) \left( \frac{x}{\lambda} - \frac{t}{T} \right), k \right]
\]

where \( K, E \) are the complete elliptic integrals of the first and second kind respectively, \( cn \) is the Jacobian elliptic...
function and $k$ is a parameter determined implicitly as a function of $H$ and $T$ by the formulae

$$ T(k) = \frac{\lambda(k)}{c(k)} $$

$$ \lambda(k) = \left( \frac{16d^3}{3H^2} \right)^{1/2} kK(k) $$

$$ c(k) = (gd)^{1/2} \left[ 1 + \frac{H}{d} k \left( \frac{1}{2} - \frac{E(k)}{K(k)} \right) \right] $$

3.3.5.6 The “Stream Function” wave theory is a purely numerical procedure for approximating a given wave profile and has a broader range of validity than the wave theories in [3.3.5.3] through [3.3.5.5]. A stream function wave solution has the general form

$$ \Psi(x, z) = c z + \sum_{n=1}^{N} X(n) \sinh nk(z + d) \cos nkx $$

where $c$ is the wave celerity and $N$ is the order of the wave theory. The required order, $N$, of the stream function theory, ranging from 1 to 10, is determined by the wave parameters $S$ and $\mu$. The closer to the breaking wave height, the more terms are required in order to give an accurate representation of the wave. Figure 3-8 shows the required order $N$ of stream function wave theory such that errors in maximum velocity and acceleration are less than one per cent.

![Figure 3-8](image)

**Figure 3-8**

**Required order $N$ of stream function wave theory**

3.3.5.7 For irregular waves, kinematics can be obtained as summation of kinematics from linear sinusoidal wave components. When the load effect under consideration is sensitive to wave kinematics in the splash zone (such as for drag loads on slender structures) a proper stretching of the kinematics profile to the wave surface should be performed (see details in DNV-RP-C205). For ULS wave loads, higher order wave models, e.g. a second-order random process, are recommended. Special consideration should be given to crest kinematics in sea states with near-breaking waves or breaking waves. Also, for shallow waters, the accuracy of linear wave theory should be assessed.
3.3.5.8 For regular waves, it is recommended to use Stokes 5th order wave theory, as long as the water depth exceeds 15% of the wavelength. The Stokes design wave should be determined based on the crest height and associated period, rather than on the wave height. In shallow waters, where the water depth is less than 15% of the wavelength, Stokes wave theory is not applicable, and other nonlinear wave theories must be applied, such as cnoidal wave theory and stream function wave theory, see [3.3.5.5] and [3.3.5.6], respectively.

3.3.6 Breaking waves

3.3.6.1 Wave breaking may take place as a result of shoaling and limited water depth. Such breaking may take place either before the waves arrive at the site or when they have arrived at the site. In both cases, the wave breaking implies that a depth-dependent limitation is imposed on the waves at the site. This depth dependency shall be taken into account when wave heights for use in design are to be determined. For this determination, the water depth corresponding to the maximum water level on the site shall be assumed. The breaking criterion is identified in Figure 3-7 and Figure 3-8. Breaking waves are irregular waves, for which the kinematics deviate from those implied by the wave theories referenced in [3.3.5.3] through [3.3.5.6]. The kinematics of breaking waves depends on the type of breaking.

3.3.6.2 There are three types of breaking waves depending on the wave steepness and the slope of the seabed:

— surging breaker
— plunging breaker
— spilling breaker.

Figure 3-9 indicates which type of breaking wave can be expected as a function of the slope of the seabed and as a function of the wave period $T$ and the wave height $H_0$ in deep waters.

![Figure 3-9](image)

**Figure 3-9**

Transitions between different types of breaking waves as a function of seabed slope, wave height in deep waters and wave period

3.4 Current

3.4.1 Current parameters

3.4.1.1 The current consists of a wind-generated current and a tidal current, and a density current when relevant.

3.4.1.2 The current is represented by the wind-generated current velocity $v_{\text{wind}0}$ at the still water level and the tidal current velocity $v_{\text{tide}0}$ at the still water level.

3.4.1.3 Other current components than wind-generated currents, tidal currents and density currents may exist. Examples of such current components are

— subsurface currents generated by storm surge and atmospheric pressure variations
— near-shore, wave-induced surf currents running parallel to the coast.

3.4.2 Current data

3.4.2.1 Current statistics are to be used as a basis for representation of the long-term and short-term current conditions. Empirical statistical data used as a basis for design must cover a sufficiently long period of time.
Guidance note:
Current data obtained on site are to be preferred over current data observed at an adjacent location. Measured current data are to be preferred over visually observed current data. Continuous records of data are to be preferred over records with gaps. Longer periods of observation are to be preferred over shorter periods.

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

3.4.2.2 The variation of the current with the water depth shall be considered when relevant.

3.4.2.3 In regions where bottom material is likely to erode, special studies of current conditions near the sea bottom may be required.

3.4.3 Current modelling

3.4.3.1 When detailed field measurements are not available, the variation in current velocity with depth may be taken as

\[ v(z) = v_{\text{tide}}(z) + v_{\text{wind}}(z) \]

where

\[ v_{\text{tide}}(z) = v_{\text{tide}0} \left( \frac{h + z}{h} \right)^{\frac{1}{7}} \]

for \( z \leq 0 \)

and

\[ v_{\text{wind}}(z) = v_{\text{wind}0} \left( \frac{h_0 + z}{h_0} \right) \]

for \( -h_0 \leq z \leq 0 \)

in which

- \( v(z) \) = total current velocity at level \( z \)
- \( z \) = vertical coordinate from still water level, positive upwards
- \( v_{\text{tide}0} \) = tidal current at still water level
- \( v_{\text{wind}0} \) = wind-generated current at still water level
- \( h \) = water depth from still water level (taken as positive)
- \( h_0 \) = reference depth for wind-generated current; \( h_0 = 50 \text{ m} \).

3.4.3.2 The variation in current profile with variation in water depth due to wave action shall be accounted for. In such cases, the current profile may be stretched or compressed vertically, such that the current velocity at any proportion of the instantaneous depth is kept constant. By this approach, the surface current component remains constant, regardless of the sea elevation during the wave action.

3.4.3.3 Unless data indicate otherwise, the wind-generated current at still water level may be estimated as

\[ v_{\text{wind}0} = k \cdot U_0 \]

where

- \( k = 0.015 \) to 0.03.
- \( U_0 \) = 1-hour mean wind speed at 10 m height.

3.5 Water Level

3.5.1 Water level parameters

3.5.1.1 The water level consists of a mean water level in conjunction with tidal water and a wind- and pressure-induced storm surge. The tidal range is defined as the range between the highest astronomical tide (HAT) and the lowest astronomical tide (LAT), see Figure 3-10. The mean water level (MWL) is defined as the average of HAT and LAT.

Guidance note:
HAT is the highest water level that can be predicted to occur under any combination of astronomical conditions, i.e. the level of high tide when all harmonic components causing the tide are in phase. LAT is the lowest water level that can be predicted to occur under any combination of astronomical conditions, i.e. the level of low tide when all harmonic components causing the tide are in phase.

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---
3.5.1.2 For design purposes either a high water level or a low water level will be governing and both need to be considered. The high water level consists of an astronomical tide above MWL plus a positive storm surge component. The low water level consists of an astronomical tide below MWL plus a negative storm surge component.

**Guidance note:**
When a high water level is governing, usually a high water level with a specified return period will be needed for design. Likewise, when a low water level is governing, usually a low water level with a specified return period will be needed for design. When the storm surge component at a location in question is insignificant and can be ignored, the water level will be governed by tide alone, and the maximum and minimum water levels to be used in design become equal to HAT and LAT, respectively.

---end---of---Guidance---note---

3.5.2 Water level data

3.5.2.1 Water level statistics are to be used as a basis for representation of the long-term and short-term water level conditions. Empirical statistical data used as a basis for design must cover a sufficiently long period of time.

**Guidance note:**
Water level data obtained on site are to be preferred over water level data observed at an adjacent location. Measured water level data are to be preferred over visually observed water level data. Continuous records of data are to be preferred over records with gaps. Longer periods of observation are to be preferred over shorter periods.

---end---of---Guidance---note---

3.5.2.2 Water level and wind are correlated, because the water level has a wind-generated component. The correlation between water level data and wind data shall be accounted for in design.

**Guidance note:**
Simultaneous observations of water level and wind data in terms of simultaneous values of water level and U_{10} should be obtained.

---end---of---Guidance---note---

3.5.3 Water level modelling

For determination of the water level for calculation of loads and load effects, both tidal water and pressure- and wind-induced storm surge shall be taken into account.

**Guidance note:**
Water level conditions are of particular importance for prediction of depth-limited wave heights.

---end---of---Guidance---note---

3.6 Ice

3.6.1 Sea ice

3.6.1.1 When the wind turbine structure is to be located in an area where ice may develop or where ice may drift, ice conditions shall be properly considered.
3.6.1.2 Relevant statistical data for the following sea ice conditions and properties shall be considered:

— geometry and nature of ice
— concentration and distribution of ice
— type of ice (ice floes, ice ridges, rafted ice etc.)
— mechanical properties of ice (compressive strength $r_u$, bending strength $r_f$)
— velocity and direction of drifting ice
— thickness of ice
— probability of encountering icebergs.

3.6.2 Snow and ice accumulation

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

3.6.2.2 Snow and ice loads due to snow and ice accumulation may be reduced or neglected if a snow and ice removal procedure is established.

3.6.2.3 Possible increases of cross-sectional areas and changes in surface roughness caused by icing shall be considered wherever relevant, when wind loads and hydrodynamic loads are to be determined.

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

3.6.3 Ice modelling

3.6.3.1 The ice thickness forms an important parameter for calculation of ice loads. The ice thickness shall be based on local ice data, e.g. as available in an ice atlas or as derived from frost index data.

3.6.3.2 As a basis for design against ice loads, the frost index $K$ may be used. The frost index for a location is defined as the absolute value of the sum of the daily mean temperature over all days whose mean temperature is less than $0^\circ C$ in one year. The frost index $K$ exhibits variability from year to year and can be represented by its probability distribution.

**Guidance note:**
Unless data indicate otherwise, the frost index may be represented by a three-parameter Weibull distribution,

$$F_K(k) = 1 - \exp(-(\frac{k-b}{a})^\beta)$$

3.6.3.3 The frost index with return period $T_R$ in units of years is defined as the $(1-1/T_R)$ quantile in the distribution of the frost index, i.e. it is the frost index whose probability of exceedance in one year is $1/T_R$. It is denoted $K_{TR}$ and is expressed as

$$K_{TR} = F_K^{-1}(1-\frac{1}{T_R})$$

3.6.3.4 The ice thickness $t$ at the end of a frost period can be estimated by

$$t = 0.032\sqrt{0.9K-50}$$

where $t$ is in units of metres and $K$ is the frost index in units of degree-days.

3.6.3.5 In near-coastal waters and in sheltered waters, such as in lakes and archipelagos, the ice sheet is normally not moving after having grown to some limiting thickness, $t_{limit}$. The limiting thickness can therefore be used to define extreme thickness events for moving ice in such waters. Unless data indicate otherwise, the limiting thickness $t_{limit}$ can be taken as the long-term mean value of the annual maximum ice thickness. No such limiting thickness is associated with moving ice in open sea, for which larger thicknesses can therefore be expected in the extreme thickness events.

**Guidance note:**
The long-term mean value of the annual maximum ice thickness may be interpreted as a measure of the ice thickness associated with a “normal winter”.

3.6.3.6 The compression strength $r_u$, the bending strength $r_f$ and the thickness of the ice may be expressed as functions of the frost index or, alternatively, in terms of their respective probability distributions. Other location-dependent parameters which may need to be considered are the floe size and the drift speed of floes.
3.6.3.7 Unless data indicate otherwise, the following general values of ice parameters apply, regardless of location:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>900 kg/m³</td>
</tr>
<tr>
<td>Unit weight</td>
<td>8.84 kN/m³</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>2 GPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.33</td>
</tr>
<tr>
<td>Ice-ice frictional coefficient</td>
<td>0.1</td>
</tr>
<tr>
<td>Ice-concrete dynamic frictional coefficient</td>
<td>0.2</td>
</tr>
<tr>
<td>Ice-steel dynamic frictional coefficient</td>
<td>0.1</td>
</tr>
</tbody>
</table>

3.7 Soil investigations and geotechnical data

3.7.1 Soil investigations

3.7.1.1 The soil investigations shall provide all necessary soil data for a detailed design. The soil investigations may be divided into geological studies, geophysical surveys and geotechnical soil investigations.

Guidance note:
A geological study, based on the geological history, can form a basis for selection of methods and extent of the geotechnical soil investigations. A geophysical survey, based on shallow seismic, can be combined with the results from a geotechnical soil investigation to establish information about soil stratification and seabed topography for an extended area such as the area covered by a wind farm. A geotechnical soil investigation consists of in-situ testing of soil and of soil sampling for laboratory testing.

---end---of---Guidance---note---

3.7.1.2 The extent of soil investigations and the choice of soil investigation methods shall take into account the type, size and importance of the wind turbine structure, the complexity of soil and seabed conditions and the actual type of soil deposits. 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.

---end---of---Guidance---note---

3.7.1.3 For multiple foundations such as in a wind farm, the soil stratigraphy and range of soil strength properties shall be assessed within each group of foundations or per foundation location, as relevant.

Guidance note:
Whether the soil stratigraphy and range of soil strength properties shall be assessed within each group of foundations or per foundation location is much a function of the degree to which the soil deposit can be considered as homogeneous. Thus, when very homogeneous soil conditions prevail, the group of foundations to be covered by such a common assessment may consist of all the 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 the wind farm are separated by kilometre-wide straits or traffic corridors. When complex or non-homogeneous soil conditions prevail, it may be necessary to limit common assessments of the soil stratigraphy and soil strength properties to cover only a few close foundations, and in the ultimate case to carry out individual assessments for individual foundations.

---end---of---Guidance---note---

3.7.1.4 Soil investigations shall provide relevant information about the soil to a depth below which possible existence of weak formations will not influence the safety or performance of the wind turbine and its support structure and foundation.

Guidance note:
For design of pile foundations against lateral loads, a combination of in-situ testing and soil borings with sampling should be carried out to sufficient depth. For slender and flexible piles in jacket type foundations, a depth to about 10 pile diameters below pile tip suffices. For less flexible monopiles with larger diameters, a depth to half a pile diameter below the assumed maximum pile penetration suffices.

For design of piles against axial loads, at least one CPT and one nearby boring should be carried out to the anticipated penetration depth of the pile plus a zone of influence. If potential end bearing layers or other dense layers, which may create driving problems, are found this scope should be increased.

For design of gravity base 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.
In seismically active areas, it may be necessary to obtain information about the shear modulus of the soil to large depths.

---end---of---Guidance---note---

3.7.1.5 Soil investigations are normally to comprise the following types of investigation:

— site geological survey
— topography survey of the seabed
— geophysical investigations for correlation with soil borings and in-situ testing
— soil sampling with subsequent static and cyclic laboratory testing
— shear wave velocity measurements for assessment of maximum shear modulus
— in-situ testing, for example by cone penetration tests (CPT), pressiometer tests and dilatometer tests.

**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 this guidance note therefore forms recommendations of a general nature which the designer, either on his own initiative or in cooperation with the classification society, 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 soil borings, suggestions for additional soil borings, and suggestions for changed positions of soil borings.

When non-homogeneous soil deposits are encountered or when difficult or weak soils are identified locally, it may be necessary to carry out more soil borings and CPTs than the tentative minimum recommended below.

For solitary wind turbine structures, one soil boring to sufficient depth for recovery of soil samples for laboratory testing is recommended as a minimum.

For wind turbine structures in a wind farm, a tentative minimum soil investigation program may contain one CPT per foundation in combination with one soil boring to sufficient depth in each corner of the area covered by the wind farm for recovery of soil samples for laboratory testing. An additional soil boring in the middle of the area will provide additional information about possible non-homogeneities over the area.

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 soil 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. Seabed samples should be taken for evaluation of scour potential.

---end---of---Guidance---note---

3.7.1.6 For further guidance and industry practice regarding requirements to scope, execution and reporting of offshore soil investigations, and to equipment, reference is made to DNV Classification Notes No. 30.4, NORSOK N-004 (App. K) and NORSOK G-001. National and international standards may be considered from case to case, if relevant.

3.7.1.7 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 classification and description
— shear strength and deformation properties, as required for the type of analysis to be carried out
— in-situ stress conditions.

The soil 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. It is of utmost importance that soil samples obtained as part of a soil investigation program are of a sufficiently good quality to allow for accurate interpretation of soil parameters for use in design. For requirements to soil sampling quality, reference is made to ISO 22475-1.

3.7.1.8 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 simple shear tests and triaxial tests are relevant types of tests for determination of strength properties.

For fibrous peats, neither direct simple shear tests nor triaxial tests are recommended for determination of strength properties. Shear strength properties of low-humified peat can be determined by ring shear tests.
3.8 Other site conditions

3.8.1 Seismicity

3.8.1.1 The level of seismic activity of the area where the wind turbine structure is to be installed shall be assessed on the basis of previous record of earthquake activity as expressed in terms of frequency of occurrence and magnitude.

3.8.1.2 For areas where detailed information on seismic activity is available, the seismicity of the area may be determined from such information.

3.8.1.3 For areas where detailed information on seismic activity is not available, the seismicity is to be determined on the basis of detailed investigations, including a study of the geological history and the seismic events of the region.

3.8.1.4 If the area is determined to be seismically active and the wind turbine structure 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 ground motion. The seismic design, including the development of the seismic design criteria for the site, shall be in accordance with recognised industry practice.

3.8.1.5 The potential for earthquake-induced sea waves, also known as tsunamis, shall be assessed as part of the seismicity assessment.

3.8.1.6 For details of seismic design criteria, reference is made to ISO 19901-2.

3.8.2 Salinity

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

Guidance note:
In estuaries 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.

3.8.3 Temperature

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

3.8.3.2 Both air and seawater temperatures are to be considered when describing the temperature environment.

3.8.4 Marine growth

3.8.4.1 The plant, animal and bacteria life on the site causes marine growth on structural components in the water and in the splash zone. The potential for marine growth shall be addressed. Marine growth adds weight to a structural component and influences the geometry and the surface texture of the component. The marine growth may hence influence the hydrodynamic loads, the dynamic response, the accessibility and the corrosion rate of the component.

Guidance note:
Marine growth can broadly be divided into hard growth and soft growth. Hard growth generally consists of animal growth such as mussels, barnacles and tubeworms, whereas soft growth consists of organisms such as hydroids, sea anemones and corals. Marine growth may also appear in terms of seaweeds and kelps. Marine organisms generally colonise a structure soon after installation, but the growth tapers off after a few years.

The thickness of marine growth depend on the position of the structural component relative to the sea level, the orientation of the component relative to the sea level and relative to the dominant current, the age of the component, and the maintenance strategy for the component.

Marine growth also depends on other site conditions such as salinity, oxygen content, pH value, current and temperature.

The corrosive environment is normally modified by marine growth in the upper submerged zone and in the lower part of the splash zone of the structural component. Depending on the type of marine growth and on other local conditions, the net effect may be either an enhancement or a retardation of the corrosion rate. Marine growth may also interfere with systems for corrosion protection, such as coating and cathodic protection.
3.8.5 Air density
Air density shall be addressed since it affects the structural design through wind loading.

3.8.6 Ship traffic
3.8.6.1 Risk associated with possible ship collisions shall be addressed as part of the basis for design of support structures for offshore wind turbines.
3.8.6.2 For service vessel collisions, the risk can be managed by designing the support structure against relevant service vessel impacts. For this purpose the limit state shall be considered as a ULS. The service vessel designs and the impact velocities to be considered are normally specified in the design basis for structural design.

3.8.7 Disposed matters
The presence of obstacles and wrecks within the area of installation shall be mapped.

3.8.8 Pipelines and cables
The presence of pipelines and cables within the area of installation shall be mapped.
SECTION 4 LOADS AND LOAD EFFECTS

4.1 Introduction

4.1.1 General

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

4.1.1.2 It is a prerequisite that the wind turbine and support structure as a minimum meet the requirements to loads given in IEC61400-1 for site-specific wind conditions.

4.2 Basis for selection of characteristic loads

4.2.1 General

4.2.1.1 Unless specific exceptions apply, as documented within this standard, the basis for selection of characteristic loads or characteristic load effects specified in [4.2.1.2] and [4.2.1.3] shall apply in the temporary and operational design conditions, respectively.

Guidance note:
Temporary design conditions cover design conditions during transport, assembly, maintenance, repair and decommissioning of the wind turbine structure.
Operational design conditions cover design conditions in the permanent phase which includes steady conditions such as power production, idling and stand-still as well as transient conditions associated with start-up, shutdown, yawing and faults.

Guidance note:
Environmental design conditions are usually specified in terms of values for quantities such as significant wave height, mean wind speed, and current velocity. In the context of marine operations, environmental design conditions are referred to as environmental design criteria.

Table 4-1 Basis for definition of characteristic loads and load effects for temporary design conditions

<table>
<thead>
<tr>
<th>Load category</th>
<th>ULS</th>
<th>FLS</th>
<th>ALS</th>
<th>SLS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permanent (G)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Variable (Q)</td>
<td>Expected value</td>
<td>Specified(^{(1)}) value</td>
<td>Specified(^{(1)}) load history</td>
<td>Specified(^{(1)}) value</td>
</tr>
<tr>
<td>Environmental (E); weather restricted</td>
<td>Specified value</td>
<td>Expected load history</td>
<td>Not applicable</td>
<td>Specified value</td>
</tr>
<tr>
<td>Environmental (E); unrestricted operations(^{(2)})</td>
<td>Based on statistical data(^{(3)})</td>
<td>Expected load history</td>
<td>Based on statistical data(^{(3,4)})</td>
<td></td>
</tr>
<tr>
<td>Accidental (A)</td>
<td></td>
<td></td>
<td>Specified value</td>
<td></td>
</tr>
<tr>
<td>Deformation (D)</td>
<td>Expected extreme value</td>
<td>Expected load history</td>
<td>Specified value</td>
<td></td>
</tr>
</tbody>
</table>

---end---of---Guidance---note---

\(^{(1)}\) The specified value or the specified load history, as applicable, shall, if relevant, be justified by calculations.
\(^{(2)}\) See DNV-OS-H101.
\(^{(3)}\) See DNV-OS-H101, Sec.3.
\(^{(4)}\) Joint probability of accident and environmental condition could be considered.

---end---of---Guidance---note---
4.2.1.3 For the operational design conditions, the basis for selection of characteristic loads and load effects specified in Table 4-2 refers to statistical terms whose definitions are given in Table 4-3.

![Table 4-2 Basis for selection of characteristic loads and load effects for operational design conditions](image)

Guidance note:
The environmental loading on support structures and foundations for wind turbines does – as far as wind loading is concerned – not always remain the way it is produced by nature, because the control system of the wind turbine interferes by introducing measures to reduce the loads.

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

4.2.1.4 Characteristic values of environmental loads or load effects, which are specified as the 98% quantile in the distribution of the annual maximum of the load or load effect, shall be estimated by their central estimates.

![Table 4-3 Statistical terms used for specification of characteristic loads and load effects](image)

4.3 Permanent loads (G)

4.3.1 General

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

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

4.3.1.2 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.

4.4 Variable functional loads (Q)

4.4.1 General

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

— personnel
— crane operational loads
— ship impacts
— loads from fendering
— loads associated with installation operations
— loads from variable ballast and equipment
— stored materials, equipment, gas, fluids and fluid pressure
— lifeboats.

4.4.1.2 For an offshore wind turbine structure, the variable functional loads usually consist of:

— actuation loads
— loads on access platforms and internal structures such as ladders and platforms
— ship impacts from service vessels
— crane operational loads.

4.4.1.3 Actuation loads result from the operation and control of the wind turbine. They are in several categories including torque control from a generator or inverter, yaw and pitch actuator loads and mechanical braking loads. In each case, it is important in the calculation of loading and response to consider the range of actuator forces available. In particular, for mechanical brakes, the range of friction, spring force or pressure as influenced by temperature and ageing shall be taken into account in checking the response and the loading during any braking event.

4.4.1.4 Actuation loads are usually represented as an integrated element in the wind turbine loads that result from an analysis of the wind turbine subjected to wind loading. They are therefore in this standard treated as environmental wind turbine loads and do therefore not appear as separate functional loads in load combinations.

4.4.1.5 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 and foundations.

4.4.1.6 Loads and dynamic factors from maintenance and service cranes on structures are to be determined in accordance with requirements given in DNV Standard for Certification No. 2.22 Lifting Appliances, latest edition.

4.4.1.7 Ship impact loads are used for the design of primary support structures and foundations and for design of some secondary structures.

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

4.4.1.9 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.

4.4.2 Variable functional loads on platform areas
Variable functional loads on platform areas of the support structure shall be based on Table 4-4 unless specified otherwise in the design basis or the design brief. 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 ship 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 4-4. 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

4.4.3 Ship impacts and collisions
4.4.3.1 Boat landings, ladders and other secondary structures in and near the water line shall be designed against operational ship impacts in the ULS. The primary structure in and near the water line shall be designed against accidental ship impacts in the ALS. Furthermore, if an accidental ship impact against a secondary structure results in larger damage of the primary structure than an accidental impact directly against the primary structure, then this load case shall also be considered in the ALS.
Guidance note:
The requirements for design against accidental ship impacts in the ALS are merely robustness requirements, which are practical to handle as ALS design. They are not really requirements for full ALS design, since designs against rare large accidental loads from impacts by larger vessels than maximum authorised service vessels are not considered, see [4.4.3.5].

---end---of---Guidance---note---

4.4.3.2 Impacts from approaching ships in the ULS shall be considered as variable functional loads. Impacts from drifting ships in the ALS shall be considered as accidental loads.

4.4.3.3 When primary structural parts such as the support structure and its foundation are exposed to ship impacts, these structural parts shall not suffer such damage that their capacities to withstand the loads they are going to be exposed to become compromised. In the ULS, secondary structural parts, such as fenders, boat landings and ladders, shall not suffer damage to such an extent that they loose their respective functions as access structures. In the ALS, 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 these supporting primary structural parts.

4.4.3.4 For design against operational ship impacts, the characteristic impact load shall be taken as the expected impact load caused by the maximum authorised service vessel approaching by bow and stern in the most severe sea state to be considered for operation of the service vessel. A vessel-specific speed shall be assumed. The speed shall not be assumed less than 0.5 m/s. Effects of wind, wave and current shall be included as well as effects of added mass, which contributes to the kinetic energy of the vessel.

Guidance note:
Data for the maximum authorised service vessel, including service vessel layout and service vessel impact velocities, are usually given in the design basis for structural design of the wind turbine structure. Data for wave, wind and current in the most severe sea state to be considered for operation of the service vessel are also usually given in the design basis. A risk analysis forms the backbone of a ship impact analysis. The expected impact load is part of the results from the risk analysis.

When specific loads are not given, the contact area can be designed by assuming an impact force

\[ F = 2.5 \cdot \Delta \]

where \( F \) is the impact force in units of kN and \( \Delta \) is the fully loaded displacement of the supply vessel in units of tons. This is based on an assumption of ship impact against a hard structure. When a damper or spring device such as a fender is provided in the area subject to the impact, a lower impact force can be used. For further background and guidance, reference is made to the DNV High Speed Light Craft Rules.

IEC61400-3 states that if no information about the service vessel is known, the impact force can normally be accounted for by applying 5 MN as a horizontal line load over the width of the support structure. This load is meant to include dynamic amplification.

---end---of---Guidance---note---

4.4.3.5 For design against accidental ship impacts, the characteristic impact load shall be taken as the impact load caused by unintended collision by the maximum authorised service vessel in daily operation. For this purpose, the service vessel shall be assumed to be drifting laterally and the speed of the drifting vessel shall be assessed. The speed shall not be assumed less than 2.0 m/s. Effects of added mass shall be included. Effects of fendering on the maximum authorised service vessel shall be considered.

Guidance note:
The maximum authorised service vessel is the largest expected vessel used in daily operation. Data for the maximum authorised service vessel, including impact velocities of a laterally drifting vessel, are usually given in the design basis for structural design of the wind turbine structure. Note that supply vessels may grow in size over the years and the accidental load may become substantial. Larger special purpose vessels used for replacement of larger components etc. should be handled by specific case-by-case safety assessments.
4.4.4 Tank pressures
Requirements to hydrostatic pressures in tanks are given in DNV-OS-C101.

4.4.5 Miscellaneous loads

4.4.5.1 Railing shall be designed for a concentrated load of 1.0 kN as well as for horizontal line load equal to 0.3 kN/m, applied to the top of the railing.

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

4.5 Environmental loads (E)

4.5.1 General

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

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

4.5.1.2 Practical information regarding environmental loads and environmental conditions is given in DNV-RP-C205.

<table>
<thead>
<tr>
<th>Table 4-4 Variable functional loads on platform areas</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local design</td>
</tr>
<tr>
<td>-------------</td>
</tr>
<tr>
<td>Distributed load $p$ (kN/m$^2$)</td>
</tr>
<tr>
<td>Storage areas</td>
</tr>
<tr>
<td>Lay down areas</td>
</tr>
<tr>
<td>Area between equipment</td>
</tr>
<tr>
<td>Walkways, staircases and external platforms</td>
</tr>
<tr>
<td>Walkways and staircases for inspection only</td>
</tr>
<tr>
<td>Internal platforms, e.g. in towers</td>
</tr>
<tr>
<td>Areas not exposed to other functional loads</td>
</tr>
</tbody>
</table>

Notes:
— Point loads are to be applied on an area 100 mm $\times$ 100 mm, and at the most severe position, but not added to wheel loads or distributed loads.
— For internal platforms, point loads are to be applied on an area 200 mm $\times$ 200 mm
— q to be evaluated for each case. Lay down areas should not be designed for less than 15 kN/m$^2$.
— $f = \min\{1.0 ; (0.5 + 3/\sqrt{A})\}$, where $A$ is the loaded area in m$^2$.
— Global load cases shall be established based upon “worst case”, characteristic load combinations, complying with the limiting global criteria to the structure. For buoyant structures these criteria are established by requirements for the floating position in still water, and intact and damage stability requirements, as documented in the operational manual, considering variable load on the deck and in tanks.
4.5.1.3 According to this standard, characteristic environmental loads and load effects shall be determined as quantiles with specified probabilities of exceedance. The statistical analysis of measured data or simulated data should make use of different statistical methods to evaluate the sensitivity of the result. The validation of distributions with respect to data should be tested by means of recognised methods. The analysis of the data shall be based on the longest possible time period for the relevant area. In the case of short time series, statistical uncertainty shall be accounted for when characteristic values are determined.

4.5.1.4 To assist in the determination of environmental loads and associated structural responses, it is recommended to carry out a coupled analysis in the time domain of the wind turbine and its support structure.

4.5.2 Wind turbine loads

4.5.2.1 Wind-generated loads on the rotor and the tower shall be considered. Wind-generated loads on the rotor and the tower include wind loads produced directly by the inflowing wind as well as indirect loads that result from the wind-generated motions of the wind turbine and the operation of the wind turbine. The direct wind-generated loads consist of

— aerodynamic blade loads (during operation, during parking and idling, during braking, and during start-up)
— aerodynamic drag forces on tower and nacelle.

The following loads, which only indirectly are produced by wind and which are a result of the operation of the wind turbine, shall be considered as wind loads in structural design according to this standard:

— gravity loads on the rotor blades, vary with time due to rotation
— centrifugal forces and Coriolis forces due to rotation
— gyroscopic forces due to yawing
— braking forces due to braking of the wind turbine.

Guidance note:
Aerodynamic wind loads on the rotor and the tower may be determined by means of aeroelastic load models. Gyroscopic loads on the rotor will occur regardless of the structural flexibility whenever the turbine is yawing during operation and will lead to a yaw moment about the vertical axis and a tilt moment about a horizontal axis in the rotor plane. For yaw speeds below 0.5 °/s gyroscopic loads can be disregarded.

4.5.2.2 For determination of wind loads, the following factors shall be considered:

— tower shadow, tower stemming and vortex shedding, which are disturbances of the wind flow owing to the presence of the tower
— wake effects wherever the wind turbine is located behind other turbines such as in wind farms
— misaligned wind flow relative to the rotor axis, e.g. owing to a yaw error
— rotational sampling, i.e. low-frequent turbulence will be transferred to high-frequent loads due to blades cutting through vortices
— aeroelastic effects, i.e., the interaction between the motion of the turbine on the one hand and the wind field on the other
— aerodynamic imbalance and rotor-mass imbalance due to differences in blade pitch
— influence of the control system on the wind turbine, for example by limiting loads through blade pitching
— turbulence and gusts
— instabilities caused by stall-induced flapwise and edgewise vibrations must be avoided
— damping
— wind turbine controller.

Guidance note:
The damping comes about as a combination of structural damping, soil damping, hydrodynamic damping and aerodynamic damping. The structural damping depends on the blade material and material in other components such as the tower. The aerodynamic damping can be determined as the outcome of an aeroelastic calculation in which correct properties for the aerodynamics are used.

The coherence of the wind and the turbulence spectrum of the wind are of significant importance for determination of tower loads such as the bending moment in the tower.

4.5.2.3 Wind turbine loads during power production and selected transient events shall be verified by load measurements that cover the intended operational range, i.e. wind speeds between cut-in and cut-out. Measurements shall be carried out by an accredited testing laboratory or the certifying body shall verify that the party conducting the testing as a minimum complies with the criteria set forth in ISO/IEC 17020 or ISO/IEC 17025, as applicable.

4.5.2.4 For design of the support structure and the foundation, a number of load cases for wind turbine loads
due to wind load on the rotor and on the tower shall be considered, corresponding to different design situations for the wind turbine. Different design situations may govern the designs of different parts of the support structure and the foundation.

The load cases shall be defined such that it is ensured that they capture the 50-year load or load effect, as applicable, for each structural part to be designed in the ULS. Likewise, the load cases shall be defined such that it is ensured that they capture all contributions to fatigue damage for design in the FLS. Finally, the load cases shall include load cases to adequately capture abnormal conditions associated with severe fault situations for the wind turbine in the ULS.

Because the wind turbine loads occur concurrently with other environmental loads such as loads from waves, current and water level, the load cases to be considered shall specify not only the wind turbine load conditions, but also their companion wave load conditions, current conditions and water level conditions.

Table 4-5 specifies a proposal for 31 load cases to consider for wind turbine load conditions and their companion wave load conditions, current conditions and water level conditions in order to fulfil the requirements in this item. The load cases in Table 4-5 refer to design in the ULS and in the FLS and include a number of abnormal load cases for the ULS.

The load cases in Table 4-5 are defined in terms of wind conditions, which are characterised by wind speed. For most of the load cases, the wind speed is defined as a particular 10-minute mean wind speed plus a particular extreme coherent gust, which forms a perturbation on the mean wind speed. Extreme coherent gusts are specified in [3.2.5.5]. Some load cases in Table 4-5 refer to the normal wind profile. The normal wind profile is given in Sec.3.

For each specified load case in Table 4-5, simulations for simultaneously acting wind and waves based on the waves given in the 4th column of Table 4-5 can be waived when it can be documented that it is not relevant to include a wave load or wave load effect for the design of a structural part in question.

Guidance note:
The 31 proposed load cases in Table 4-5 correspond to 31 load cases defined in IEC61400-3 on the basis of the load cases in IEC61400-1.

Wind load case 1.4 is usually only relevant for design of the top of the tower, and wave loading may only in rare cases have an impact on the design of this structural part.

For analysis of the dynamic behaviour of the wind turbine and its support structure for concurrently acting wind and waves, it is important to carry out the analysis using time histories of both wind and waves or relevant dynamic amplification factors should be applied to a constant wind speed or individual wave height.

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---
<table>
<thead>
<tr>
<th>Design situation</th>
<th>Load case</th>
<th>Wind condition: Wind climate ( (U_{10, \text{hub}}) ) or wind speed ( (U_{\text{hub}}) )</th>
<th>Wave condition: Sea state ( (H_S) ) or individual wave height ( (H) ) to combine with</th>
<th>Wind and wave directionality</th>
<th>Current</th>
<th>Water level</th>
<th>Other conditions</th>
<th>Limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Power production</td>
<td>1.1 NTM (9) ( v_{\text{in}} &lt; U_{10, \text{hub}} &lt; v_{\text{out}} )</td>
<td>NSS ( H_S = E[H_S</td>
<td>U_{10, \text{hub}}] )</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>For prediction of extreme loads on RNA and interface to tower</td>
<td>ULS</td>
</tr>
<tr>
<td>1.2 NTM (9) ( v_{\text{in}} &lt; U_{10, \text{hub}} &lt; v_{\text{out}} )</td>
<td>NSS ( H_S = E[H_S</td>
<td>U_{10, \text{hub}}] )</td>
<td>Codirectional in multiple directions (See [4.6.9])</td>
<td>(5)</td>
<td>Range between upper and lower 1-year water level</td>
<td>FLS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.3 ETM (9) ( v_{\text{in}} &lt; U_{10, \text{hub}} &lt; v_{\text{out}} )</td>
<td>NSS ( H_S = E[H_S</td>
<td>U_{10, \text{hub}}] )</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>ULS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.4 ECD ( U_{10, \text{hub}} = v_r - 2 \text{ m/s}, v_r, v_r + 2 \text{ m/s} )</td>
<td>NSS ( H_S = E[H_S</td>
<td>U_{10, \text{hub}}] ) ( \text{or NWH} ) ( H = E[H_S</td>
<td>U_{10, \text{hub}}] ) (3)</td>
<td>Misaligned</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>ULS</td>
<td></td>
</tr>
<tr>
<td>1.5 EWS (9) ( v_{\text{in}} &lt; U_{10, \text{hub}} &lt; v_{\text{out}} )</td>
<td>NSS ( H_S = E[H_S</td>
<td>U_{10, \text{hub}}] ) ( \text{or NWH} ) ( H = E[H_S</td>
<td>U_{10, \text{hub}}] ) (3)</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>ULS</td>
<td></td>
</tr>
<tr>
<td>1.6a NTM ( v_{\text{in}} &lt; U_{10, \text{hub}} &lt; v_{\text{out}} )</td>
<td>NSS ( H_S = H_{S,50-\text{yr}} ) (See [4.6.7.3])</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>1-year water level (4)</td>
<td>ULS</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.6b NTM ( v_{\text{in}} &lt; U_{10, \text{hub}} &lt; v_{\text{out}} )</td>
<td>SWH ( H = H_{S,50-\text{yr}} ) (See [4.6.7.3])</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>1-year water level (4)</td>
<td>ULS</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 4-5 Proposed load cases combining various environmental conditions

<table>
<thead>
<tr>
<th>Design situation</th>
<th>Load case</th>
<th>Wind condition: Wind climate ($U_{10,hub}$) or wind speed ($V_{hub}$)</th>
<th>Wave condition: Sea state ($H_S$) or individual wave height ($H$) to combine with</th>
<th>Wind and wave directionality</th>
<th>Current</th>
<th>Water level</th>
<th>Other conditions</th>
<th>Limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Power production plus occurrence of fault</td>
<td><strong>2.1</strong> NTM $v_{in} &lt; U_{10,hub} &lt; v_{out}$</td>
<td>NSS $H_S = E[H_S</td>
<td>U_{10,hub}]$</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>Control system fault or loss of electrical connection</td>
<td>ULS</td>
</tr>
<tr>
<td></td>
<td><strong>2.2</strong> NTM $v_{in} &lt; U_{10,hub} &lt; v_{out}$</td>
<td>NSS $H_S = E[H_S</td>
<td>U_{10,hub}]$</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>Protection system fault or preceding internal electrical fault</td>
<td>ULS Abnormal</td>
</tr>
<tr>
<td></td>
<td><strong>2.3a</strong> EOG $U_{10,hub} = v_{out}$ and $v_t \pm 2$ m/s</td>
<td>NSS $H_S = E[H_S</td>
<td>U_{10,hub}]$ or NWH $H = E[H_S</td>
<td>U_{10,hub}]$ (3) (6)</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>External or internal electrical fault including loss of electrical network connection</td>
</tr>
<tr>
<td></td>
<td><strong>2.3b</strong> NTM (8) $v_{in} &lt; U_{10,hub} &lt; v_{out}$</td>
<td>NSS $H_S = E[H_S</td>
<td>U_{10,hub}]$</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>External or internal electrical fault including loss of electrical network connection</td>
<td>ULS</td>
</tr>
<tr>
<td></td>
<td><strong>2.4</strong> NTM $v_{in} &lt; U_{10,hub} &lt; v_{out}$</td>
<td>NSS $H_S = E[H_S</td>
<td>U_{10,hub}]$</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>Control or protection system fault including loss of electrical network</td>
<td>FLS</td>
</tr>
<tr>
<td>Start up</td>
<td><strong>3.1</strong> NWP $v_{in} &lt; U_{10,hub} &lt; v_{out}$ + normal wind profile to find average vertical wind shear across swept area of rotor</td>
<td>NSS $H_S = E[H_S</td>
<td>U_{10,hub}]$ or NWH $H = E[H_S</td>
<td>U_{10,hub}]$ (3)</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>Range between upper and lower 1-year water level</td>
</tr>
<tr>
<td></td>
<td><strong>3.2</strong> EOG $U_{10,hub} = v_{in}$, $v_{out}$ and $v_t \pm 2$ m/s</td>
<td>NSS $H_S = E[H_S</td>
<td>U_{10,hub}]$ or NWH $H = E[H_S</td>
<td>U_{10,hub}]$ (3)</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>Range between upper and lower 1-year water level</td>
</tr>
<tr>
<td></td>
<td><strong>3.3</strong> EDC $U_{10,hub} = v_{in}$, $v_{out}$ and $v_t \pm 2$ m/s</td>
<td>NSS $H_S = E[H_S</td>
<td>U_{10,hub}]$ or NWH $H = E[H_S</td>
<td>U_{10,hub}]$ (3)</td>
<td>Misaligned</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>Range between upper and lower 1-year water level</td>
</tr>
<tr>
<td>Normal shutdown</td>
<td><strong>4.1</strong> NWP $v_{in} &lt; U_{10,hub} &lt; v_{out}$ + normal wind profile to find average vertical wind shear across swept area of rotor</td>
<td>NSS $H_S = E[H_S</td>
<td>U_{10,hub}]$ or NWH $H = E[H_S</td>
<td>U_{10,hub}]$ (3)</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td>Range between upper and lower 1-year water level</td>
</tr>
<tr>
<td></td>
<td><strong>4.2</strong> EOG $U_{10,hub} = v_{out}$ and $v_t \pm 2$ m/s</td>
<td>NSS $H_S = E[H_S</td>
<td>U_{10,hub}]$ or NWH $H = E[H_S</td>
<td>U_{10,hub}]$ (3)</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td></td>
</tr>
</tbody>
</table>
### Table 4-5: Proposed load cases combining various environmental conditions

<table>
<thead>
<tr>
<th>Design situation</th>
<th>Load case</th>
<th>Wind condition: Wind climate ($U_{10,hub}$) or wind speed ($U_{hub}$)</th>
<th>Wave condition: Sea state ($H_S$) or individual wave height ($H$) to combine with</th>
<th>Wind and wave directionality</th>
<th>Current</th>
<th>Water level</th>
<th>Other conditions</th>
<th>Limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Emergency shutdown</td>
<td>5.1 NTM</td>
<td>$U_{10,hub} = v_{out} + v_i \pm 2$ m/s</td>
<td>$H_S = E[H_S</td>
<td>U_{10,hub}]$</td>
<td>Codirectional in one direction</td>
<td>Wind-generated current</td>
<td>MWL</td>
<td></td>
</tr>
<tr>
<td>Parked (standing still or idling)</td>
<td>6.1a EWM</td>
<td>Turbulent wind $U_{10,hub} = U_{10,50-yr}$ (characteristic standard deviation of wind speed $\sigma_{U,c} = 0.11 \cdot U_{10,hub}$)</td>
<td>$H_S = H_{S,50-yr}$ (1)</td>
<td>Misaligned Multiple directions</td>
<td>50-year current</td>
<td>50-year water level</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.1b EWM</td>
<td>Steady wind $U_{10,hub} = 1.4 \cdot U_{10,50-yr}$</td>
<td>$RWH = \psi \cdot H_{50-yr}$ (2)</td>
<td>Misaligned Multiple directions</td>
<td>50-year current</td>
<td>50-year water level</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.1c RWM</td>
<td>Steady wind $U_{10,hub} = 1.1 \cdot U_{10,50-yr}$</td>
<td>$EWH = H_{50-yr}$</td>
<td>Misaligned Multiple directions</td>
<td>50-year current</td>
<td>50-year water level</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.2a EWM</td>
<td>Turbulent wind $U_{10,hub} = U_{10,50-yr}$ (characteristic standard deviation of wind speed $\sigma_{U,c} = 0.11 \cdot U_{10,hub}$)</td>
<td>$H_S = H_{S,50-yr}$ (1)</td>
<td>Misaligned Multiple directions</td>
<td>50-year current</td>
<td>50-year water level</td>
<td>Loss of electrical network connection</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.2b EWM</td>
<td>Steady wind $U_{10,hub} = 1.4 \cdot U_{10,50-yr}$</td>
<td>$RWH = \psi \cdot H_{50-yr}$ (2)</td>
<td>Misaligned Multiple directions</td>
<td>50-year current</td>
<td>50-year water level</td>
<td>Loss of electrical network connection</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.3a EWM</td>
<td>Turbulent wind $U_{10,hub} = U_{10,1-yr}$ (characteristic standard deviation of wind speed $\sigma_{U,c} = 0.11 \cdot U_{10,hub}$)</td>
<td>$H_S = H_{S,1-yr}$ (1)</td>
<td>Misaligned Multiple directions</td>
<td>1-year current</td>
<td>1-year water level</td>
<td>Extreme yaw mis-alignment</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.3b EWM</td>
<td>Steady wind $U_{10,hub} = 1.4 \cdot U_{10,1-yr}$</td>
<td>$RWH = \psi \cdot H_{1-yr}$ (2)</td>
<td>Misaligned Multiple directions</td>
<td>1-year current</td>
<td>1-year water level</td>
<td>Extreme yaw mis-alignment</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.4 NTM</td>
<td>$U_{10,hub} &lt; 0.7U_{10,50-yr}$</td>
<td>NSS $H_S$ according to joint probability distribution of $H_S$, $T_p$ and $U_{10,hub}$</td>
<td>Codirectional in multiple direction (See [4.6.9])</td>
<td>(5)</td>
<td>Range between upper and lower 1-year water level</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 4.5 Proposed load cases combining various environmental conditions

<table>
<thead>
<tr>
<th>Design situation</th>
<th>Load case</th>
<th>Wind condition: Wind climate ($U_{10,\text{hub}}$) or wind speed ($U_{\text{hub}}$)</th>
<th>Wave condition: Sea state ($H_S$) or individual wave height ($H$) to combine with</th>
<th>Wind and wave directionality</th>
<th>Current</th>
<th>Water level</th>
<th>Other conditions</th>
<th>Limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parked and fault conditions</td>
<td>7.1a</td>
<td>EWM Turbulent wind $U_{10,\text{hub}} = U_{10,\text{1-yr}}$ (characteristic standard deviation of wind speed $\sigma_U = 0.11 \cdot U_{10,\text{hub}}$)</td>
<td>ESS $H_S = H_{S,1-yr}$ (1)</td>
<td>Misaligned Multiple directions</td>
<td>1-year current</td>
<td>1-year water level</td>
<td></td>
<td>ULS Abnormal</td>
</tr>
<tr>
<td></td>
<td>7.1b</td>
<td>EWM Steady wind $U_{\text{hub}} = 1.4 \cdot U_{10,\text{1-yr}}$</td>
<td>RWH $H = \psi \cdot H_{1-yr}$ (2)</td>
<td>Misaligned Multiple directions</td>
<td>1-year current</td>
<td>1-year water level</td>
<td></td>
<td>ULS Abnormal</td>
</tr>
<tr>
<td></td>
<td>7.1c</td>
<td>RWM Steady wind $U_{\text{hub}} = 0.88 \cdot U_{10,50-yr}$</td>
<td>EWH $H = H_{1-yr}$</td>
<td>Misaligned Multiple directions</td>
<td>1-year current</td>
<td>1-year water level</td>
<td></td>
<td>ULS Abnormal</td>
</tr>
<tr>
<td></td>
<td>7.2</td>
<td>NTM $U_{10,\text{hub}} &lt; 0.7U_{10,50-yr}$</td>
<td>NSS $H_S$ according to joint probability distribution of $H_S$, $T_p$ and $U_{10,\text{hub}}$</td>
<td>Codirectional in multiple direction (See [4.6.9])</td>
<td>(5)</td>
<td>Range between upper and lower 1-year water level</td>
<td></td>
<td>FLS</td>
</tr>
</tbody>
</table>

- **EWM**: Extreme wind conditions
- **RWH**: Random wave height
- **ESS**: Exceedance standard value
- **NSS**: Non-exceedance standard value
- **NTM**: Nominal wind conditions
- **U**: Wind speed
- **H**: Wave height
- **$U_{10,\text{hub}}$**: 10-year hub wind speed
- **$U_{\text{hub}}$**: Hub wind speed
- **$H_{S,1-yr}$**: 1-year significant wave height
- **$H_{1-yr}$**: 1-year wave height
- **$\psi$**: Wave directionality parameter
- **$\sigma_U$**: Standard deviation of wind speed
- **$T_p$**: Wave period
- **$\text{ULS}$**: Ultimate limit state
- **$\text{FLS}$**: Fatigue limit state

*Note: The equations and conditions listed above are excerpts from the Offshore Standard DNV-OS-J101, May 2014.*
### Table 4-5  Proposed load cases combining various environmental conditions

<table>
<thead>
<tr>
<th>Design situation</th>
<th>Load case</th>
<th>Wind condition: Wind climate ($U_{10, hub}$) or wind speed ($U_{hub}$)</th>
<th>Wave condition: Sea state ($H_S$) or individual wave height ($H$) to combine with</th>
<th>Wind and wave directionality</th>
<th>Current</th>
<th>Water level</th>
<th>Other conditions</th>
<th>Limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transport, assembly, maintenance and repair</td>
<td>8.2a</td>
<td>EWM Steady wind $U_{hub} = 1.4 \cdot U_{10,1-yr}$</td>
<td>RWH $H = \psi \cdot H_{1-yr} (2)$</td>
<td>Codirectional in one direction</td>
<td>1-year current</td>
<td>1-year water level</td>
<td></td>
<td>ULS Abnormal</td>
</tr>
<tr>
<td></td>
<td>8.2b</td>
<td>RWM Steady wind $U_{hub} = 0.88 \cdot U_{10,50-yr}$</td>
<td>EWH $H = H_{1-yr}$</td>
<td>Codirectional in one direction</td>
<td>1-year current</td>
<td>1-year water level</td>
<td></td>
<td>ULS Abnormal</td>
</tr>
<tr>
<td></td>
<td>8.3</td>
<td>NTM $U_{10,hub} &lt; 0.7U_{10,50-yr}$ NEG Hs according to joint probability distribution of $H_S$, $T_p$ and $U_{10,hub}$</td>
<td>Codirectional in multiple direction (See [4.6.9])</td>
<td>(5)</td>
<td>Range between upper and lower 1-year water level</td>
<td></td>
<td>FLS</td>
<td></td>
</tr>
</tbody>
</table>

1) In cases where load and response simulations are to be performed and the simulation period is shorter than the reference period for the significant wave height $H_S$, the significant wave height needs to be converted to a reference period equal to the simulation period, see [3.3.2.2]. Moreover, an inflation factor on the significant wave height needs to be applied in order to make sure that the shorter simulation period captures the maximum wave height when the original reference period does. When the reference period is 3 hours and the simulation period is 1 hour, the combined conversion and inflation factor is 1.09 provided the wave heights are Rayleigh-distributed and the number of waves in 3 hours is 1000. Likewise, if the simulation period is longer than the averaging period for the mean wind speed, a deflation factor on $U_{10}$ may be applied. When the simulation period is 1 hour and the averaging period is 10 minutes, the deflation factor may be taken as 0.95.

2) It is practice for offshore structures to apply $\psi = H_{5-yr}/H_{50-yr}$, where $H_{5-yr}$ and $H_{50-yr}$ denote the individual wave heights with 5- and 50-year return period, respectively. The shallower the water depth, the larger is usually the value of $\psi$.

3) The load case is not driven by waves and it is optional whether the wind load shall be combined with an individual wave height or with a sea state.

4) The water level shall be taken as the upper-tail 50-year water level in cases where the extreme wave height will become limited by the water depth.

5) In principle, current acting concurrently with the design situation in question needs to be included, because the current influences the hydrodynamic coefficients and thereby the fatigue loading relative to the case without current. However, in many cases current will be of little importance and can be ignored, e.g. when the wave loading is inertia-dominated or when the current speed is small.

6) In the case that the extreme operational gust is combined with an individual wave height rather than with a sea state, the resulting load shall be calculated for the most unfavourable location of the profile of the individual wave relative to the temporal profile of the gust.

7) Whenever the wave loading associated with a specific load case refers to a wave train or a time series of wave loads, the sought-after combined load effect shall be interpreted as the maximum resulting load effect from the time series of load effects which is produced by the simulations.

8) Load case 2.3b is an optional alternative to load case 2.3a. For each considered value of $U_{10}$, 12 response simulations shall be carried out. A nominal extreme response is evaluated as the mean of 12 responses plus three times the standard deviation of the responses. The characteristic response for load case 2.3b is determined as the extreme value among the nominal extreme responses obtained for the range of $U_{10}$ investigated.

9) Inside large wind farms, the characteristic standard deviation $\sigma_{U_{a,c}}$ of the ambient wind speed inside the wind farm shall be used instead of the characteristic standard deviation $\sigma_{U_{c}}$. 
4.5.2.5 Analysis of the load cases in Table 4-5 shall be carried out for assumptions of aligned wind and waves or misaligned wind and waves, or both, as relevant. Analysis of the load cases in Table 4-5 shall be carried out for assumptions of wind in one single direction or wind in multiple directions, as relevant.

4.5.2.6 9 of the 31 load cases specified in Table 4-5 define abnormal load cases to be considered for loads and load effects due to wind loading on the rotor and the tower in the ULS. Abnormal load cases are wind load cases associated with a number of severe fault situations for the wind turbine, which result in activation of system protection functions. Abnormal load cases are in general less likely to occur than the normal load cases considered for the ULS in Table 4-5.

4.5.2.7 Computer codes which are used for prediction of wind turbine loads shall be validated for the purpose. The validation shall be documented.

4.5.2.8 Table 4-5 refers to two turbulence models, viz. the normal turbulence model NTM and the extreme turbulence model ETM. By the NTM the characteristic value $\sigma_{U,C}$ of the standard deviation $\sigma_U$ of the wind speed shall be taken as the 90% quantile in the probability distribution of $\sigma_U$ conditional on $U_{10\text{hub}}$. By the ETM the characteristic value $\sigma_{U,C}$ of the standard deviation $\sigma_U$ of the wind speed shall be taken as the value of $\sigma_U$ which together with $U_{10\text{hub}}$ forms a combined $(U_{10\text{hub}}, \sigma_U)$ event with a return period of 50 years.

**Guidance note:**

When available turbulence data are insufficient to establish the characteristic standard deviation $\sigma_U$ of the wind speed, the following expressions may be applied for this standard deviation for the normal and extreme turbulence models, respectively:

$$
\sigma_{U,C,NTM} = I_{ref} \cdot (0.75U_{10\text{hub}} + b)
$$

$$
\sigma_{U,C,ETM} = c \cdot I_{ref} \cdot (0.072 \cdot \left(\frac{V_{ave}}{c} + 3\right) \cdot \left(\frac{U_{10\text{hub}}}{c} - 4\right) + 10)
$$

in which $I_{ref}$ is a reference turbulence intensity defined as the expected turbulence intensity at a 10-minute mean wind speed of 15 m/s, $V_{ave}$ is the annual average wind speed at hub height, $b = 5.6$ m/s and $c = 2$ m/s.

The expressions are based on probability distribution assumptions which do not account for wake effects in wind farms. The expressions are therefore not valid for design of wind turbine structures for locations whose extreme turbulences are governed by wake effects.

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

4.5.2.9 The wind turbine loads in items [4.5.2.1] through [4.5.2.8] do not apply to meteorological masts nor to other structures which do not support wind turbines. For such structures, wind loads, which have not been filtered through a wind turbine to form wind turbine loads, shall be considered. Wind loads on meteorological masts may be calculated according to EN 1991-1-4. Load combinations where these wind loads are combined with other types of environmental loads can be taken according to DNV-OS-C101.

**Guidance note:**

Detailed methods for calculation of wind loads on meteorological masts are given in DIN 4131 and DIN 4133.

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

4.5.3 Determination of characteristic hydrodynamic loads

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

4.5.3.2 Hydrodynamic model tests should be carried out to:

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

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

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

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

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

4.5.4.1 For calculation of wave loads, a recognised wave theory for representation of the wave kinematics shall be applied. The wave theory shall be selected with due consideration of the water depth and of the range of validity of the theory.

4.5.4.2 Methods for wave load prediction shall be applied that properly account for the size, shape and type of structure.

4.5.4.3 For slender structures, such as jacket structure components and monopile structures, Morison’s equation can be applied to calculate the wave loads.

4.5.4.4 For large volume structures, for which the wave kinematics are disturbed by the presence of the structure, wave diffraction analysis shall be performed to determine local (pressure force) and global wave loads. For floating structures wave radiation forces must be included.

4.5.4.5 Both viscous effects and potential flow effects may be important in determining the wave-induced loads on a wind turbine support structure. Wave diffraction and radiation are included in the potential flow effects.

**Guidance note:**

Figure 4-1 can be used as a guidance to establish when viscous effects or potential flow effects are important. Figure 4-1 refers to horizontal wave-induced forces on a vertical cylinder, which stands on the seabed and penetrates the free water surface, and which is subject to incoming regular waves.

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

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

**Figure 4-1**

Relative importance of inertia, drag and diffraction wave forces

4.5.4.6 Wave forces on slender structural members, such as a cylinder submerged in water, can be predicted by Morison’s equation. By this equation, the horizontal force on a vertical element \( dz \) of the structure at level \( z \) is expressed as:

\[
dF = dF_M + dF_D = C_M \rho \pi \frac{D^2}{4} \ddot{x} dz + C_D \rho \frac{D}{2} |\dot{x}| \dot{x} dz
\]

where the first term is an inertia force and the second term is a drag force. Here, \( C_D \) and \( C_M \) are drag and inertia coefficients, respectively, \( D \) is the diameter of the cylinder, \( \rho \) is the density of water, \( \ddot{x} \) is the horizontal wave-induced velocity of water, and \( \dot{x} \) is the horizontal wave-induced acceleration of water. The level \( z \) is measured from still water level, and the \( z \) axis points upwards. Thus, at seabed \( z = -d \), when the water depth is \( d \).

**Guidance note:**

The drag and inertia coefficients are in general functions of the Reynolds number, the Keulegan-Carpenter number and the relative roughness. The coefficient also depends on the cross-sectional shape of the structure and of the orientation of the body. For a cylindrical structural member of diameter \( D \), the Reynolds number is defined as \( Re = u_{\text{max}} D / \nu \) and the Keulegan-Carpenter number as \( KC = u_{\text{max}} T_1 / D \), where \( u_{\text{max}} \) is the maximum horizontal particle velocity at still water level, \( \nu \) is the kinematic viscosity of seawater, and \( T_1 \) is the intrinsic period of the waves. Re and
KC, and in turn $C_D$ and $C_M$, may attain different values for the extreme waves that govern the ULS and for the moderate waves that govern the FLS.

The drag coefficient $C_{DS}$ for steady-state flow can be used as a basis for calculation of $C_D$ and $C_M$. The drag coefficient $C_{DS}$ for steady-state flow depends on the roughness of the surface of the structural member and may be taken as

$$C_{DS} = \begin{cases} 
0.65 & \text{for } k / D < 10^{-4} \text{ (smooth)} \\
29 + 4 \log_{10}(k / D) & \text{for } 10^{-4} < k / D < 10^{-2} \\
1.05 & \text{for } k / D > 10^{-2} \text{ (rough)}
\end{cases}$$

in which $k$ is the surface roughness and $D$ is the diameter of the structural member. New uncoated steel and painted steel can be assumed to be smooth. For concrete and highly rusted steel, $k = 0.003$m can be assumed. For marine growth, $k = 0.005$ to 0.05$m$ can be assumed.

The drag coefficient $C_D$ depends on $C_{DS}$ and on the KC number and can be calculated as

$$C_D = C_{DS} \cdot \psi(C_{DS}, KC)$$

in which the wake amplification factor $\psi$ can be read off from Figure 4-2. For intermediate roughnesses between smooth and rough, linear interpolation is allowed between the curves for smooth and rough cylinder surfaces in Figure 4-2.

![Figure 4-2](image)

**Figure 4-2**

*Wake amplification factor as function of KC number for smooth (solid line) and rough (dotted line)*

For KC $< 3$, potential theory is valid with $C_M = 2.0$. For KC $> 3$, the inertia coefficient $C_M$ can be taken as

$$C_M = \max\{2.0 - 0.044(KC - 3); 1.6 - (C_{DS} - 0.65)\}$$

where $C_{DS}$ depends on the surface roughness of the structural member as specified above.

As an example, in 30 to 40 metres of water in the southern and central parts of the North Sea, $C_D = 0.8$ and $C_M = 1.6$ can be applied for diameters less than 2.2$m$ for use in load calculations for fatigue limit states.

For structures in shallow waters near coastlines where there is a significant current in addition to the waves, $C_M$ should not be taken less than 2.0.

For long waves in shallow water, the depth variation of the water particle velocity is usually not large. Hence it is recommended to use force coefficients based on the maximum horizontal water particle velocity $u_{\text{max}}$ at the free surface.

When waves are asymmetric, which may in particular be the case in shallow waters, the front of the wave has a different steepness than the rear of the wave. Since the wave force on a structure depends on the steepness of the wave, caution must be exercised to apply the asymmetric wave to the structure in such a manner that the wave load impact is calculated from that of the two wave steepnesses which will produce the largest force on the structure.

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

4.5.4.7 The resulting horizontal force $F$ on the cylinder can be found by integration of Morison’s equation for values of $z$ from $-d$ to the wave crest, $\eta(t)$. 
Guidance note:

For non-breaking waves, the resulting horizontal force becomes

\[ F = F_M + F_D \]

\[ = \int_{-d}^{\eta(t)} C_M \rho \frac{D^2}{4} \eta(t) \, dz \]

\[ + \int_{-d}^{\eta(t)} C_D \rho \frac{D}{2} \eta(t) \, dz \]

The integration from \(-d\) to 0 ignores contributions to the force from the wave crest above the still water level at \(z = 0\). This is a minor problem when the inertia force \(F_M\) is the dominating force component in \(F\), since \(F_M\) has its maximum when a nodal line at the still water level passes the structure. The drag force \(F_D\) has its maximum when the crest or trough passes the structure. If this force is the dominating force component in \(F\), a significant error can be introduced by ignoring the contribution from the wave crest.

The relative magnitude between the inertia force component \(F_M\) and the drag force component \(F_D\) can be expressed by the ratio between their amplitudes, \(A = A_M/A_D\). Figure 4-2 can be used to quickly establish whether the inertia force or the drag force is the dominating force, once the ratios \(H/D\) and \(d/\lambda\) have been calculated. Structures which come out above the curve marked \(A = 1\) in Figure 4-2 experience drag-dominated loads, whereas structures which come out below this curve experience inertia-dominated loads.

Morison’s equation is only valid when the dimension of the structure is small relative to the wave length, i.e. when \(D < 0.2\lambda\). The integrated version of Morison’s equation given here is only valid for non-breaking waves. However, Morison’s equation as formulated for a vertical element \(dz\) is valid for calculation of wave forces from both breaking and non-breaking waves as long as the element is fully submerged. In deep water, waves break when \(H/\lambda\) exceeds about 0.14. In shallow water, waves break when \(H/d\) exceeds about 0.78.

Figure 4-2 is based on linear wave theory and should be used with caution, since linear wave theory may not always be an adequate wave theory as a basis for prediction of wave forces in particularly shallow waters. 5th order stream function theory is usually considered the best wave theory for representation of wave kinematics in shallow waters. For prediction of wave forces for fatigue assessment, higher order stream function theory can be applied for water depths less than approximately 15 m, whereas Stokes 5th order theory is recommended for water depths in excess of approximately 30 m.

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

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

**Figure 4-3**
Relative magnitude of inertia and drag forces for cylinders with \(D/\lambda < 0.2\)
4.5.4.8 When the dimension of the structure is large compared with the wave length, typically when \( D > 0.2 \lambda \), Morison’s equation is not valid. The inertia force will be dominating and can be predicted by diffraction theory.

**Guidance note:**

For linear waves, the maximum horizontal force on a vertical cylinder of radius \( R = D/2 \) installed in water of depth \( d \) and subjected to a wave of amplitude \( A_W \), can be calculated as

\[
F_{x,\text{max}} = \frac{4 \rho g A \sinh k(d + A \sin \alpha)}{k^2} \xi
\]

and its arm measured from the seabed is

\[
h_F = d \left( \frac{k d \sinh[kd] - \cosh[kd] + 1}{kd \sinh[kd]} \right)
\]

The coefficients \( \xi \) and \( \alpha \) are given in Table E2.

The diffraction solution for a vertical cylinder given above is referred to as the MacCamy-Fuchs solution. The terms given represents essentially a corrected inertia term which can be used in Morison’s equation together with the drag term.

The formulae given in this guidance note are limited to vertical circular cylinders with constant diameter \( D \). For other geometries of the support structure, such as when a conical component is present in the wave-splash zone to absorb or reduce ice loads, diffraction theory is still valid, but the resulting force and moment arm will come out different from the vertical cylinder solutions given here.

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

4.5.4.9 For evaluation of load effects from wave loads, possible ringing effects shall be included in the considerations. When a steep, high wave encounters a monopile, high frequency nonlinear wave load components can coincide with natural frequencies of the structure causing resonant transient response in the global bending modes of the pile. Such ringing effects are only of significance in combination with extreme first order wave frequency effects. Ringing should be evaluated in the time domain with due consideration of higher order wave load effects. The magnitude of the first ringing cycles is governed by the magnitude of the wave impact load and its duration is related to the structural resonance period.

**Guidance note:**

Ringing can occur if the lowest natural frequencies of the structure do not exceed three to four times the typical wave frequency. In case the natural frequency exceeds about five to six times \( f_p \), where \( f_p \) denotes the peak frequency, ringing can be ruled out. When a dynamic analysis is carried out, any ringing response will automatically appear as part of the results from the analysis, provided the wave forces are properly modelled and included in the analysis.

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---
4.5.4.10 When waves are likely to break on the site of the structure or in its vicinity, wave loads from breaking waves shall be considered in the design of the structure. Wave loads from breaking waves depend on the type of breaking waves. A distinction is made between surging, plunging and spilling waves. The kinematics is different for these three types of breaking waves.

Guidance note:
For plunging waves, an impact model may be used to calculate the wave forces on a structure. The impact force from a plunging wave can be expressed as

$$ F = \frac{1}{2} \rho C_S A u^2 $$

where \( u \) denotes the water particle velocity in the plunging wave crest, \( \rho \) is the mass density of the fluid, \( A \) is the area on the structure which is assumed exposed to the slamming force, and \( C_S \) is the slamming coefficient. For a smooth circular cylinder, the slamming coefficient should not be taken less than 3.0. The upper limit for the slamming coefficient is \( 2 \pi \). Careful selection of slamming coefficients for structures can be made according to DNV-RP-C205. The area \( A \) exposed to the slamming force depends on how far the plunging breaker has come relative to the structure, i.e., how wide or pointed it is when it hits the structure. Plunging waves are rare in Danish and German waters.

For a plunging wave that breaks immediately in front of a vertical cylinder of diameter \( D \), the duration \( T \) of the impact force on the cylinder is given by

$$ T = \frac{13D}{64c} $$

where \( c \) denotes the wave celerity.

For surging and spilling waves, an approach to calculate the associated wave forces on a vertical cylindrical structure of diameter \( D \) can be outlined as follows: The cylinder is divided into a number of sections. As the breaking wave approaches the structure, the instantaneous wave elevation close to the cylinder defines the time instant when a section is hit by the wave and starts to penetrate the sloping water surface. The instantaneous force per vertical length unit on
this section and on underlying sections, which have not yet fully penetrated the sloping water surface, can be calculated as

\[ f = \frac{1}{2} \rho C_S D u^2 \]

where \( u \) denotes the horizontal water particle velocity, \( \rho \) is the mass density of the fluid, and \( C_S \) is the slamming coefficient whose value can be taken as

\[ C_S = 5.15 \left( \frac{D}{D + 19s} + \frac{0.107s}{D} \right) \]

for \( 0 < s < D \).

The penetration distance \( s \) for a section in question is the horizontal distance from the periphery on the wet side of the cylinder to the sloping water surface, measured in the direction of the wave propagation. For fully submerged sections of the cylinder, the wave forces can be determined from classical Morison theory with mass and drag terms using constant mass and drag coefficients,

\[ f = \rho \pi C_M \frac{D^2}{4} \frac{du}{dt} + \frac{1}{2} \rho C_D D u^2 \]

The water particle velocity \( u \) is to be determined from the wave kinematics for the particular type of breaking wave in question.

4.5.4.11 Computer codes which are used for prediction of wave loads on wind turbine structures shall be validated for the purpose. The validation shall be documented.

4.5.4.12 Characteristic extreme wave loads are in this standard defined as wave load values with a 50-year return period.

**Guidance note:**

In the southern and central parts of the North Sea, experience shows that the ratio between the 100- and 50-year wave load values \( F_{\text{wave},100}/F_{\text{wave},50} \) attains a value approximately equal to 1.10. Unless data indicate otherwise, this value of the ratio \( F_{\text{wave},100}/F_{\text{wave},50} \) may be applied to achieve the 50-year wave load \( F_{\text{wave},50} \) in cases where only the 100-year value \( F_{\text{wave},100} \) is available, provided the location in question is located in the southern or central parts of the North Sea.

4.5.4.13 Any walkways or platforms mounted on the support structure of an offshore wind turbine shall be located above the splash zone.

4.5.4.14 For prediction of wave loading, the effect of disturbed water particle kinematics due to secondary structures shall be accounted for. Disturbed kinematics due to large volume structures should be calculated by a wave diffraction analysis. For assessment of shielding effects due to multiple slender structures reference is made to DNV-RP-C205.

4.5.5 Air gap

4.5.5.1 For determination of the deck elevation of access platforms which are not designed to resist 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.

4.5.5.2 The air gap shall fulfil the following requirements:

- The air gap shall be at least 1.0 m when installation tolerances, global water level rise (due to global warming) and extreme water level are included in the calculation of the total extreme sea elevation in addition to the design wave crest.
- The air gap shall be at least 20% of the 50-year significant wave height. Installation tolerances and global water level rise are covered by this value and need not be included in the calculation of the total extreme sea elevation.

**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.

It is also important to consider run-up, i.e. water pressed upwards along the surface of the structure or the structural members that support the access platform, 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.
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 (See IEC 61400-3).

---end of Guidance note---

4.5.5.3 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.

4.5.6 Ice loads

4.5.6.1 Ice loads on offshore support structures, caused by laterally moving ice, are in general difficult to assess. They depend much on the nature and quality of the ice, including the age of the ice, the salinity of the ice and the temperature of the ice. The younger the ice and the less saline, the stronger is the ice and the larger are the forces induced by the moving ice. Large uncertainties are involved and caution must therefore be exercised when ice loads are to be predicted. Information about ice loads can be found in ISO 19906.

4.5.6.2 Loads from laterally moving ice shall be based on relevant full scale measurements, on model experiments which can be reliably scaled, or on recognised theoretical methods. When determining the magnitude and direction of ice loads, consideration is to be given to the nature of the ice, the mechanical properties of the ice, the ice-structure contact area, the size and shape of the structure, and the direction of the ice movements. The oscillating nature of the ice loads, including build-up and fracture of moving ice, is to be considered.

Guidance note:
Theoretical methods for calculation of ice loads should always be used with caution.

In sheltered waters and in waters close to the coastline, a rigid ice cover will usually not move once it has grown to exceed some limiting thickness, see [3.6.3.5]. In such land-locked waters, loads caused by moving ice may be calculated on the basis of this limiting thickness only, while loads associated with thermal pressures, arch effects and vertical lift need to be calculated on the basis of the actual characteristic ice thickness as required by this standard.

In open sea, where moving ice can be expected regardless of thickness, all ice loads shall be based on the actual characteristic ice thickness as required by this standard.

---end of Guidance note---

4.5.6.3 Where relevant, ice loads other than those caused by laterally moving ice are to be taken into account. Such ice loads include, but are not limited to, the following:

— loads due to rigid covers of ice, including loads due to arch effects and water level fluctuations
— loads due to masses of ice frozen to the structure
— pressures from pack ice and ice walls
— thermal ice pressures associated with temperature fluctuations in a rigid ice cover
— possible impact loads during thaw of the ice, e.g. from falling blocks of ice
— loads due to icing and ice accretion.

Guidance note:
Owing to the very large forces associated with pack ice, it is not recommended to install wind turbines in areas where pack ice may build up.

---end of Guidance note---

4.5.6.4 Table 4-6 specifies a proposal for 7 load cases to consider for ice load conditions and their companion wind load conditions in order to fulfill the requirements in [4.5.5.1] and [4.5.5.2]. The load cases in Table 4-6 refer to design in the ULS and in the FLS. The load cases for design in the ULS are based on a characteristic ice thickness \( t_C \) equal to the 50-year ice thickness \( t_{50} \) or equal to the limiting thickness \( t_{\text{limit}} \), depending on location.

4.5.6.5 Wherever there is a risk that falling blocks of ice may hit a structural member, a system to protect these members from the falling ice shall be arranged.

4.5.6.6 Possible increases in volume due to icing are to be considered when wind and wave loads acting on such volumes are to be determined.

4.5.6.7 The structure shall be designed for horizontal and vertical static ice loads. Frictional coefficients between ice and various structural materials are given in [3.6.3.7]. Ice loads on vertical structures may be determined according to API RP2N.

Guidance note:
Horizontal loads from moving ice should be considered to act in the same direction as the concurrent wind loads.

Unilateral thermal ice pressures due to thermal expansion and shrinkage can be assumed to act from land outwards toward the open sea or from the centre of a wind farm radially outwards. Larger values of unilateral thermal ice...
pressures will apply to stand-alone structures and to the peripheral structures of a wind farm than to structures in the interior of a wind farm.

The water level to be used in conjunction with calculation of ice loads shall be taken as the high water level or the low water level with the required return period, whichever is the most unfavourable.

### Table 4-6  Proposed load cases combining ice loading and wind loading

<table>
<thead>
<tr>
<th>Design situation</th>
<th>Load case</th>
<th>Ice condition</th>
<th>Wind condition: Wind climate ($U_{10\text{hub}}$)</th>
<th>Water level</th>
<th>Other conditions</th>
<th>Limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Power production</td>
<td>E1</td>
<td>Horizontal load due to temperature fluctuations</td>
<td>$v_{\text{in}} &lt; U_{10\text{hub}} &lt; v_{\text{out}} + \text{NTM}$</td>
<td>1-year water level</td>
<td>ULS</td>
<td>1-year water level</td>
</tr>
<tr>
<td></td>
<td>E2</td>
<td>Horizontal load due to water level fluctuations or arch effects</td>
<td>$v_{\text{in}} &lt; U_{10\text{hub}} &lt; v_{\text{out}} + \text{NTM}$</td>
<td>1-year water level</td>
<td>ULS</td>
<td>1-year water level</td>
</tr>
<tr>
<td></td>
<td>E3</td>
<td>Horizontal load from moving ice floe</td>
<td>Ice thickness: $I_C = I_{50}$ in open sea, $I_C = I_{\text{limit}}$ in land-locked waters</td>
<td>$v_{\text{in}} &lt; U_{10\text{hub}} &lt; v_{\text{out}} + \text{ETM}$</td>
<td>50-year water level</td>
<td>For prediction of extreme loads</td>
</tr>
<tr>
<td></td>
<td>E4</td>
<td>Horizontal load from moving ice floe</td>
<td>Ice thickness: $I_C = I_{50}$ in open sea, $I_C = I_{\text{limit}}$ in land-locked waters</td>
<td>$v_{\text{in}} &lt; U_{10\text{hub}} &lt; v_{\text{out}}$</td>
<td>1-year water level</td>
<td>FLS</td>
</tr>
<tr>
<td></td>
<td>E5</td>
<td>Vertical force from fast ice covers due to water level</td>
<td>No wind load applied</td>
<td>1-year water level</td>
<td>ULS</td>
<td>1-year water level</td>
</tr>
<tr>
<td>Parked (standing still or idling)</td>
<td>E6</td>
<td>Pressure from hummocked ice and ice ridges</td>
<td>Turbulent wind $U_{10\text{hub}} = U_{10,50\text{-yr}} + \text{characteristic standard deviation of wind speed } \sigma_{U_c} = 0.11 \times U_{10\text{hub}}$</td>
<td>1-year water level</td>
<td>ULS</td>
<td>1-year water level</td>
</tr>
<tr>
<td></td>
<td>E7</td>
<td>Horizontal load from moving ice floe</td>
<td>Ice thickness: $I_C = I_{50}$ in open sea, $I_C = I_{\text{limit}}$ in land-locked waters</td>
<td>$U_{10\text{hub}} &lt; 0.7U_{10,50\text{-yr}} + \text{NTM}$</td>
<td>1-year water level</td>
<td>FLS</td>
</tr>
</tbody>
</table>

4.5.6.8 Ice loads on inclined structural parts such as ice-load reducing cones in the splash zone may be determined according to Ralston’s formulae. Ralston’s formulae are given in App.L.

**Guidance note:**

To achieve an optimal ice cone design and avoid that ice load governs the design of the support structure and foundation, it is recommended to adjust the inclination angle of the cone such that the design ice load is just less than the design wave load.

For ice-load reducing cones of the “inverted cone” type that will tend to force moving ice downwards, the bottom of ice-load reducing cones is recommended to be located a distance of at least one ice thickness below the water level.

The flexural strength of ice governs the ice loads on inclined structures. Table 4-7 specifies values of the flexural strength for various return periods in different waters.

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

### Table 4-7  Flexural strength of sea ice

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Flexural strength of ice, $r_f$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Southern North Sea, Skagerrak, Kattegat</td>
</tr>
<tr>
<td>5</td>
<td>0.25</td>
</tr>
<tr>
<td>10</td>
<td>0.39</td>
</tr>
<tr>
<td>50</td>
<td>0.50</td>
</tr>
<tr>
<td>100</td>
<td>0.53</td>
</tr>
</tbody>
</table>
4.5.6.9 Unless data indicate otherwise, the characteristic local ice pressure for use in design against moving ice can be taken as

\[ r_{local,C} = r_{u,C} \left( 1 + \frac{t_C^2}{A_{local}} \right) \]

where \( r_{u,C} \) is the characteristic compressive strength of the ice, \( t_C \) is the characteristic thickness of the ice, and \( A_{local} \) is the area over which the local ice pressure is applied.

**Guidance note:**
The characteristic compressive strength of ice depends on local conditions such as the salinity. For load cases, which represent rare events, the characteristic compressive strength is expressed in terms of a required return period. Table 4-8 specifies values of the compressive strength for various return periods in different waters.

For load cases, which are based on special events during thaw, break-up and melting, lower values than those associated with rare events during extreme colds apply. 1.5 MPa applies to rigid ice during spring at temperatures near the melting point. 1.0 MPa applies to partly weakened, melting ice at temperatures near the melting point.

Local values for the characteristic ice thickness \( t_C \) shall be applied.

4.5.6.10 The structure shall be designed for horizontal and vertical dynamic ice loads.

**Guidance note:**
For structures located in areas, where current is prevailing, the dynamic ice load may govern the design when it is combined with the concurrent wind load. This may apply to the situation when the ice breaks in the spring.

4.5.6.11 The level of application of horizontal ice load depends on the water level and the possible presence of ice-load reducing cones. Usually a range of application levels needs to be considered.

4.5.6.12 When ice breaks up, static and dynamic interactions will take place between the structure and the ice. For structures with vertical walls, the natural vibrations of the structure will affect the break-up frequency of the ice, such that it becomes tuned to the natural frequency of the structure. This phenomenon is known as lock-in and implies that the structure becomes excited to vibrations in its natural mode shapes. The structure shall be designed to withstand the loads and load effects from dynamic ice loading associated with lock-in when tuning occurs. All contributions to damping in the structure shall be considered. Additional damping owing to pile-up of ice floes may be accounted for when it can be documented.

4.5.6.13 The criterion for occurrence of tuning is

\[ \frac{U_{ice}}{t \cdot f_n} > 0.3 \]

where \( U_{ice} \) is the velocity of the ice floe, \( t \) is the thickness of the ice, and \( f_n \) is the natural frequency of the structure.

As long as the total structural damping is not too small, the following method for analysis of dynamic ice loading can be applied:

The loading can be assumed to follow a serrated profile in the time domain as shown in Figure 4-4. The maximum value of the load shall be set equal to the static horizontal ice load. After crushing of the ice, the loading is reduced to 20% of the maximum load. The load is applied with a frequency that corresponds to the natural frequency of the structure. All such frequencies that fulfill the tuning criterion shall be considered.

For structures with too small total damping, caution must be exercised as this method may underestimate dynamic amplifications and thereby lead to nonconservative results for the design.
Guidance note:
In assessing whether the structural damping is too small, it may be helpful to consider that, based on field experience, structures with natural frequencies in the range 0.4 to 10 Hz have experienced lock-in vibrations when the total structural damping has been lower than 3% of critical damping.

---end---of---Guidance---note---

4.5.6.14 For conical structures, the break-up frequency of the ice shall be assumed independent of the natural vibrations of the structure. It shall be assured in the design that the frequency of the ice load is not close to the natural frequency of the structure.

Guidance note:
The frequency of the ice load can be determined as

\[ f_{\text{ice}} = \frac{U_{\text{ice}}}{L} \]

where \( U_{\text{ice}} \) is the velocity of the ice floe, and \( L \) is the crack length in the ice.

The force can be applied according to the simplified model in Figure 4-4, even though the failure mechanism in the ice is different for conical structures than for vertical structures. For prediction of the crack length \( L \), the following two models are available:

1) \( L = \frac{1}{2} \rho D \),

\[ \text{where } D \text{ is the diameter of the cone at the water table and } \rho \text{ is determined from Figure 4-5 as a function of } \gamma_W D^2 / (\sigma_f t), \text{ in which } \sigma_f \text{ is the flexural strength of the ice, } \gamma_W \text{ is the unit weight of water and } t \text{ is the thickness of the ice.} \]

2) \[ L = \left( \frac{1}{2} \frac{E t^3}{12 \gamma_W (1 - \nu^2)} \right)^{0.25} \]

where \( E \) is Young’s modulus of the ice and \( \nu \) is Poisson’s ratio of the ice.

Neither of these formulae for prediction of \( L \) reflects the dependency of \( L \) on the velocity of the ice floe, and the formulae must therefore be used with caution. The prediction of \( L \) is in general rather uncertain, and relative wide ranges for the frequency \( f_{\text{ice}} \) must therefore be assumed in design to ensure that an adequate structural safety is achieved.

---end---of---Guidance---note---
4.5.7 Water level loads

4.5.7.1 Tidal effects and storm surge effects shall be considered in evaluation of responses of interest. Higher water levels tend to increase hydrostatic loads and current loads on the structure; however, situations may exist where lower water levels will imply the larger hydrodynamic loads. Higher mean water levels also imply a decrease in the available airgap to access platforms and other structural components which depend on some minimum clearance.

Guidance note:
In general, both high water levels and low water levels shall be considered, whichever is most unfavourable, when water level loads are predicted.

For prediction of extreme responses, there are thus two 50-year water levels to consider, viz. a low 50-year water level and a high 50-year water level. Situations may exist where a water level between these two 50-year levels will produce the most unfavourable responses.

4.5.8 Earthquake loads

4.5.8.1 When a wind turbine structure is to be designed for installation on a site which may be subject to an earthquake, the structure shall be designed to withstand the earthquake loads. Response spectra in terms of so-called pseudo response spectra may be used for this purpose.

Guidance note:
Pseudo response spectra for a structure are defined for displacement, velocity and acceleration. For a given damping ratio $\gamma$ and angular frequency $\omega$, the pseudo response spectrum $S$ gives the maximum value of the response in question over the duration of the response. This can be calculated from the ground acceleration history by Duhamel’s integral. The following pseudo response spectra are considered:

- $S_D$, response spectral displacement
- $S_V$, response spectral velocity
- $S_A$, response spectral acceleration.

For a lightly damped structure, the following approximate relationships apply, $S_A = \omega^2 S_D$ and $S_V = \omega S_D$, such that it suffices to establish the acceleration spectrum and then use this to establish the other two spectra.

It is important to analyse the wind turbine structure for the earthquake-induced accelerations in one vertical and two horizontal directions. It usually suffices to reduce the analysis in two horizontal directions to an analysis in one horizontal direction, due to the symmetry of the dynamic system. The vertical acceleration may lead to buckling in the tower. Since there is not expected to be much dynamics involved with the vertical motion, the tower may be analysed with respect to buckling for the load induced by the maximum vertical acceleration caused by the earthquake. However, normally the only apparent buckling is that associated with the ground motion in the two horizontal directions, and the buckling analysis for the vertical motion may then not be relevant. For detailed buckling analysis for the tower, reference is made to DNV-OS-C101 and NORSOK.

For analysis of the horizontal motions and accelerations, the wind turbine can be represented by a concentrated mass on top of a vertical rod, and the response spectra can be used directly to determine the horizontal loads set up by the ground motions. For a typical wind turbine, the concentrated mass can be taken as the mass of the nacelle, including the rotor mass, plus ¼ of the tower mass.
4.5.8.2 When a wind turbine structure is to be installed in areas which may be subject to tsunamis set up by earthquakes, the load effect of the tsunamis on the structure shall be considered.

Guidance note:
Tsunamis are seismic sea waves. To account for load effects of tsunamis on wind turbine structures in shallow waters, an acceptable approach is to calculate the loads for the maximum sea wave that can exist on the site for the given water depth.

---end of Guidance note---

4.5.9 Marine growth

4.5.9.1 Marine growth shall be taken into account by increasing the outer diameter of the structural member in question in the calculations of hydrodynamic wave and current loads. The thickness of the marine growth depends on the depth below sea level and the orientation of the structural component. The thickness shall be assessed based on relevant local experience and existing measurements. Site-specific studies may be necessary in order to establish the likely thickness and depth dependence of the growth.

Guidance note:
Unless data indicate otherwise, the following marine growth profile may be used for design in Norwegian and UK waters:

<table>
<thead>
<tr>
<th>Depth below MWL (m)</th>
<th>Marine growth thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Central and Northern North Sea (56° to 59°N)</td>
</tr>
<tr>
<td>–2 to 40</td>
<td>100</td>
</tr>
<tr>
<td>&gt; 40</td>
<td>50</td>
</tr>
</tbody>
</table>

Somewhat higher values, up to 150 mm between sea level and LAT –10 m, may be seen in the Southern North Sea. Offshore central and southern California, marine growth thicknesses of 200 mm are common.

In the Gulf of Mexico, the marine growth thickness may be taken as 38 mm between LAT+3 m and 50 m depth, unless site-specific data and studies indicate otherwise.

Offshore West Africa, the marine growth thickness may be taken as 100 mm between LAT and 50 m depth and as 300 mm in the splash zone above LAT, unless data indicate otherwise.

The outer diameter of a structural member subject to marine growth shall be increased by twice the recommended thickness at the location in question.

The type of marine growth may have an impact on the values of the hydrodynamic coefficients that are used in the calculations of hydrodynamic loads from waves and current.

---end of Guidance note---

4.5.9.2 Due to the uncertainties involved in the assumptions regarding the marine growth on a structure, a strategy for inspection and possible removal of the marine growth should be planned as part of the design of the structure. When such a strategy is planned, the inspection frequency, the inspection method and the criteria for growth removal shall be based on the impact of the marine growth on the safety of the structure and on the extent of experience with marine growth under the specific conditions prevailing at the site.

4.5.10 Scour

4.5.10.1 Scour is the result of erosion of soil particles at and near a foundation and is caused by waves and current. Scour is a load effect and may have an impact on 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 are given in [10.2.3].

4.5.10.2 Means to prevent scour and requirements to such means are given in [10.2.3].

4.5.11 Loads during load-out, transport and installation

4.5.11.1 Design criteria shall be defined for acceptable environmental conditions during load-out (from shore), transportation, installation and dismantling of offshore wind turbine structures and their foundations. Design criteria shall also be defined for acceptable environmental conditions during installation, dismantling and replacement of wind turbine rotors and nacelles because these temporary phases for wind turbine components influence the support structures and foundations, e.g. by exposing them to temporary loads. Based on the applied working procedures, on the vessels used and on the duration of the operation in question, acceptable limits for the following environmental properties shall be specified:

— wind speed and wind direction
— significant wave height and wave direction
— wave height and wave period
— water level (tide)

---end of Guidance note---
Guidance note:
Both the significant wave height and the heights of individual waves are usually needed. The significant wave height is used in the calculations prior to the marine operation, but can hardly be used during the operation itself. During the operation itself, the individual wave heights are used, based on an on-site assessment.
Water level is usually only an issue on locations with significant tide.
Ice may be an issue for maintenance and repair operations in harsher climates.

4.5.11.2 Based on the chosen environmental design criteria, relevant temporary load conditions shall be identified, and the resulting dynamic characteristic loads acting on the structural components shall be determined.
These load conditions will normally include, but are not limited to, the following loads:

a) for the load-out phase:
   - global loads in structure during onshore and inshore lifting and overturning (upending) operations
   - local loads in structural lifting details, such as padeyes and trunnions.

b) for the transport phase:
   - global loads in structure supported by transport supports and seafastening elements
   - local loads in the component which are in the way of vertical and horizontal supports.

c) for the installation phase:
   - global loads in structure during offshore lifting and upending, if relevant
   - local loads in structural lifting details lifting global loads in structure during lifting and overturning (reverse upending).

4.5.11.3 The characteristic dynamic loads shall be combined with associated static loads, using the principles as given in Sec.5, in order to obtain the design loads for the temporary phases.

4.5.11.4 DNV Rules for Planning and Execution of Marine Operations gives requirements and guidelines for how to determine environmental design criteria for temporary phases.

4.5.11.5 For a general presentation of marine operations, the concept of Marine Warranty Survey and a presentation of the DNV Rules for Planning and Execution of Marine Operations, reference is made to Sec.12.

4.6 Combination of environmental loads

4.6.1 General

4.6.1.1 This section gives requirements for combination of environmental loads in the operational condition.

4.6.1.2 The requirements refer to characteristic wind turbine loads based on an investigation of the load cases specified in Tables E1 and E3.

4.6.1.3 For design against the ULS, the characteristic environmental load effect shall be taken as the 98% quantile in the distribution of the annual maximum environmental load effect, i.e. it is the load effect whose return period is 50 years, and whose associated probability of exceedance is 0.02. When the load effect is the result of the simultaneous occurrence of two or more environmental load processes, these requirements to the characteristic load effect apply to the combined load effect from the concurrently acting load processes. The subsequent items specify how concurrently acting environmental loads can be combined to arrive at the required characteristic combined load effect.

4.6.1.4 Environmental loads are loads exerted by the environments that surround the structure. Typical environments are wind, waves, current, and ice, but other environments may also be thought of such as temperature and ship traffic. Each environment is usually characterized by an intensity parameter. Wind is usually characterized by the 10-minute mean wind speed, waves by the significant wave height, current by the mean current, and ice by the ice thickness.

4.6.2 Environmental states

Environmental states are defined as short-term environmental conditions of approximately constant intensity parameters. The typical duration of an environmental state is 10 minutes or one hour. The long-term variability of multiple intensity parameters representative of multiple, concurrently active load environments can be represented by a scattergram or by a joint probability distribution function including information about load direction.
4.6.3 Environmental contours
An environmental contour is a contour drawn through a set of environmental states on a scattergram or in a joint probability density plot for the intensity parameters of concurrently active environmental processes. The environmental states defined by the contour are states whose common quality is that the probability of a more rare environmental state is \( p = T_S / T_R \) where \( T_S \) is the duration of the environmental state and \( T_R \) is a specified return period.

Guidance note:
The idea of the environmental contour is that the environmental state whose return period is \( T_R \) is located somewhere along the environmental contour defined based on \( T_R \). When only one environmental process is active, the environmental contour reduces to a point on a one-dimensional probability density plot for the intensity parameter of the process in question, and the value of the intensity in this point becomes equal to the value whose return period is \( T_R \).

For an offshore wind turbine, the wind process and the wave process are two typical concurrent environmental processes. The 10-minute mean wind speed \( U_{10} \) represents the intensity of the wind process, and the significant wave height \( H_S \) represents the intensity of the wave process. The joint probability distribution of \( U_{10} \) and \( H_S \) can be represented in terms of the cumulative distribution function \( f_{U_{10}} \) for \( U_{10} \) and the cumulative distribution function \( f_{H_S} \) for \( H_S \) conditional on \( U_{10} \). A first-order approximation to the environmental contour for return period \( T_R \) can be obtained as the infinite number of solutions \((U_{10}, H_S)\) to the following equation:

\[
\sqrt{\left(\Phi^{-1}(F_{U_{10}}(U_{10}))\right)^2 + \left(\Phi^{-1}(F_{H_S|U_{10}}(H_S))\right)^2} = \Phi^{-1}(1 - \frac{T_S}{T_R})
\]

valid for \( T_S < T_R \)

in which \( \Phi^{-1} \) denotes the inverse of the standard normal cumulative distribution function.

The environmental contour whose associated return period is 50 years is useful for finding the 50-year load effect in the wind turbine structure when the assumption can be made that the 50-year load effect occurs during the 50-year environmental state. When this assumption can be made, the 50-year load effect can be estimated by some high quantile, such as the 90\% quantile, in the conditional distribution of the load effect in that environmental state among the environmental states of duration \( T_S \) along the 50-year environmental contour that produces the largest load effect.

The environmental state is characterized by a specific duration, e.g. one hour. Whenever data for \( U_{10} \) and \( H_S \) refer to reference periods which are different from this duration, appropriate conversions of these data to the specified environmental state duration must be carried out.

4.6.4 Combined load and load effect due to wind load and wave load

4.6.4.1 In a short-term period with a combination of waves and fluctuating wind, the individual variations of the two load processes about their respective means can be assumed uncorrelated. This assumption can be made regardless of the intensities and directions of the load processes, and regardless of possible correlations between intensities and between directions.

4.6.4.2 Two methods for combination of wind load and wave load are given in this standard:

— Linear combination of wind load and wave load, or of wind load effects and wave load effects, see [4.6.5].

— Combination of wind load and wave load by simulation, see [4.6.6].

4.6.4.3 The load combination methods presented in [4.6.5] and [4.6.6] and the load combinations specified in [4.6.7] are expressed in terms of combinations of wind load effects, wave load effects and possible other load effects. This corresponds to design according to Approach (1) in [2.5.2.2]. For design according to Approach (2) in [2.5.2.2], the term “load effect” in [4.6.5], [4.6.6] and [4.6.7] shall be interpreted as “load” such that “design loads” are produced by the prescribed combination procedures rather than “design load effects”. Following Approach (2), the design load effects then result from structural analyses for these design loads.

4.6.5 Linear combinations of wind load and wave load

The combined load effect in the structure due to concurrent wind and wave loads may be calculated by combining the separately calculated wind load effect and the separately calculated wave load effect by linear superposition. This method may be applied to concept evaluations and in some cases also to load calculations for final design, for example in shallow water or when it can be demonstrated that there is no particular dynamic effect from waves, wind, ice or combinations thereof.

According to the linear combination format presented in Sec.2, the design combined load effect is expressed as:

\[
S_d = \gamma_1 S_{\text{wind},k} + \gamma_2 S_{\text{wave},k}
\]

in which \( S_{\text{wind},k} \) denotes the characteristic wind load effect and \( S_{\text{wave},k} \) denotes the characteristic wave load effect. It is a prerequisite for using this approach to determine the design combined load effect that the separately calculated value of the characteristic wave load effect \( S_{\text{wind},k} \) is obtained for realistic assumptions.
about the equivalent damping that results from structural damping, soil damping, hydrodynamic damping and aerodynamic damping. The equivalent damping depends on the following conditions related to the wind turbine and the wind load on the turbine:

— whether the wind turbine is exposed to wind or not
— whether the wind turbine is in operation or is parked
— whether the wind turbine is a pitch-regulated turbine or a stall-regulated turbine
— the direction of the wind loading relative to the direction of the wave loading.

Correct assumptions for the wind turbine and the wind load shall be made according to this list. The equivalent damping shall be determined in correspondence with these assumptions. Structural analyses by an adequate structural analysis model and based on this equivalent damping shall then be used to determine the characteristic wave load effect $S_{\text{wave},k}$. The damping from the wind turbine should preferably be calculated directly in an integrated model.

**Guidance note:**

When the characteristic load effect $S_{\text{wave},k}$ is defined as the load effect whose return period is $T_R$, the determination of $S_{\text{wave},k}$ as a quantile in the distribution of the annual maximum load effect may prove cumbersome and involve a large number of structural analyses to be carried out before contributions to this distribution from all important sea states have been included.

When the assumption can be made that $S_{\text{wave},k}$ occurs during the particular sea state of duration $T_S$ whose significant wave height $H_k$ has a return period equal to $T_R$, then $S_{\text{wave},k}$ may be estimated by the expected value of the maximum load effect during this sea state, and the analytical efforts needed may become considerably reduced. The assumption that $S_{\text{wave},k}$ occurs in the sea state whose return period is $T_R$ is often reasonable, unless sea states exist for which the structure becomes more dynamically excited than by this particular sea state, for example sea states involving wave trains whose periods are close to integer multiples of the natural period of the structure, and sea states with peak periods close to integer multiples of the natural period.

When the structural analysis involves executions of a number of simulations of the maximum load effect that occurs during the sea state whose significant wave height has a return period $T_R$, then $S_{\text{wave},k}$ shall be estimated by the mean of these simulated maximum load effects.

The wind loads in the wind direction during idling and with the yaw system in function will be quite small and will consist mainly of drag on the tower and the nacelle cover. During this condition it is implied that the blades are pitched such that the blade profiles point in the direction up against the wind or in the wind direction. The largest wind loads in this condition will be the blade loads that act perpendicular to the wind direction.

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

### 4.6.6 Combination of wind load and wave load by simulation

The combined load effect in the structure due to concurrent wind and wave loads may alternatively be calculated by direct simulation. This approach is based on structural analyses in the time domain for simultaneously applied simulated time series of the wind load and the wave load. By this approach, simulated time series of the combined load effect results, from which the characteristic combined load effect $S_k$ is interpreted.

**Guidance note:**

The approach requires that a global structural analysis model is established, e.g. in the form of a beam-element based frame model, to which load series from several simultaneously acting load processes can be applied. Although this is here exemplified for two concurrently acting load processes, viz. wind loads and wave loads, this can be generalised to include other concurrent load processes.

When the characteristic load effect $S_k$ is defined as the load effect whose return period is $T_R$, the determination of $S_k$ as a quantile in the distribution of the annual maximum load effect may prove cumbersome and involve a large number of structural analyses to be carried out before contributions to this distribution from all important environmental states have been included.

When the assumption can be made that $S_k$ occurs during an environmental state of duration $T_S$ associated with a return period $T_R$, then $S_k$ may be estimated by the expected value of the maximum load effect during such an environmental state, and the analytical efforts needed may become considerably reduced. Under this assumption, $S_k$ can be estimated by the expected value of the maximum load effect that can be found among the environmental states on the environmental contour whose associated return period is $T_R$.

To simulate one realisation of the maximum load effect along the environmental contour whose associated return period is $T_R$, one structural simulation analysis is carried out for each environmental state along the environmental contour and one maximum load effect results for each one of these states. The same seed needs to be applied for each environmental state investigated this way. A following search along the contour will identify the sought-after realisation of the maximum load effect. In practice, it will suffice to carry out structural simulation analyses only for a limited number of environmental states along a part of the environmental contour. The procedure is repeated for a number of different seeds, and a corresponding number of maximum load effect realisations are obtained. The sought-after characteristic load effect $S_k$ is estimated by the mean of these simulated maximum load effects.

When dynamic simulations utilising a structural dynamics model are used to calculate load effects, the total period of load effect data simulated shall be long enough to ensure statistical reliability of the estimate of the sought-after maximum load effect. At least six ten-minute stochastic realisations (or a continuous 60-minute period) shall be
required for each 10-minute mean, hub-height wind speed considered in the simulations. Since the initial conditions used for the dynamic simulations typically have an effect on the load statistics during the beginning of the simulation period, the first 5 seconds of data (or longer if necessary) shall be eliminated from consideration in any analysis interval involving turbulent wind input.

The wind loads in the wind direction during idling and with the yaw system in function will be quite small and will consist mainly of drag on the tower and the nacelle cover. During this condition it is implied that the blades are pitched such that the blade profiles point in the direction up against the wind or in the wind direction. The largest wind loads in this condition will be the blade loads that act perpendicular to the wind direction.

The wave field must be simulated by applying a valid wave theory according to Sec. 3. Simulation using linear wave theory (Airy theory) in shallow waters may significantly underestimate the wave loads.

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

4.6.7 Basic load cases

4.6.7.1 When information is not available to produce the required characteristic combined load effect directly, the required characteristic combined load effect can be obtained by combining the individual characteristic load effects due to the respective individual environmental load types. Table 4-9 specifies a list of load cases that shall be considered when this approach is followed, thereby to ensure that the required characteristic combined load effect, defined as the combined load effect with a return period of 50 years, is obtained for the design. Each load case is defined as the combination of two or more environmental load types. For each load type in the load combination of a particular load case, the table specifies the characteristic value of the corresponding separately determined load effect. The characteristic value is specified in terms of the return period.

Table 4-9 Proposed load combinations for load calculations according to [4.6.5]

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Load combination</th>
<th>Environmental load type and return period to define characteristic value of corresponding load effect</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS</td>
<td>1</td>
<td>Wind: 50 years; Waves: 5 years; Current: 5 years; Ice: 50 years; Water level: 50 years</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Wind: 5 years; Waves: 50 years; Current: 5 years; Ice: 50 years; Water level: 50 years</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Wind: 5 years; Waves: 5 years; Current: 50 years; Ice: 50 years; Water level: 50 years</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Wind: 5 years; Waves: 5 years; Current: 50 years; Ice: 50 years; Water level: Mean water level</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Wind: 5 years; Waves: 5 years; Current: 50 years; Ice: 50 years; Water level: Mean water level</td>
</tr>
</tbody>
</table>

Guidance note:
Table 4-9 forms the basis for determination of the design combined load effect according to the linear combination format in [4.6.5]. Table 4-9 refers to a characteristic combined load effect with a return period of 50 years and shall be used in conjunction with load factors specified in Sec. 5.

When it can be assumed that a load effect whose return period is $T_R$ occurs during the environmental state whose return period is $T_R$, then the tabulated recurrence values in Table 4-9 can be used as the return period for the load intensity parameter for the load type that causes the particular load effect in question. With this interpretation, Table 4-9 may be used as the basis for determination of the characteristic combined load effect by linear combination, in which case the analyses for the particular load cases of Table 4-9 replace the more cumbersome searches for the characteristic load effect on environmental contours as described in [4.6.3].

When the direction of the loading is an important issue, it may be of particular relevance to maintain that the return periods of Table 4-9 refer to load effects rather than to load intensities.

For determination of the 50-year water level, two values shall be considered, viz. the high water level which is the 98% quantile in the distribution of the annual maximum water level and the low water level which is the 2% quantile in the distribution of the annual minimum water level. For each load combination in Table 4-9, the most unfavourable value among the two shall be used for the 50-year water level.

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

4.6.7.2 Every time a load combination is investigated, which contains a load effect contribution from wind load, the load combination shall be analysed for two different assumptions about the state of the wind turbine:

— wind turbine in operation (power production)
— parked wind turbine (idling or standing still).

The largest load effect resulting from the corresponding two analyses shall be used for design.

Guidance note:
It will usually not be clear beforehand which of the two assumptions will produce the largest load effect, even if the blades of the parked turbine are put in the braking position to minimise the wind loads.

In a ULS situation where the characteristic wind load effect is to be taken as the 50-year wind load effect, the calculation for the wind turbine in operation will correspond to calculation of the load effect for a wind climate whose
intensity is somewhere between the cut-in wind speed and the cut-out wind speed. For stall-regulated wind turbines, the cut-out wind speed dominates the extreme operational forces. For pitch-regulated wind turbines, the extreme operational forces occur for wind climates whose intensities are near the 10-minute mean wind speed where regulation starts, typically 13 to 14 m/s.

For the parked wind turbine, the calculation in a ULS situation will correspond to the calculation of the 50-year wind load effect as if the wind turbine would be in the parked condition during its entire design life.

4.6.7.3 When it can be established as unlikely that the wind turbine will be in operation during the wave, ice, current and/or water level conditions that form part of a load combination under investigation, the requirement of [4.6.7.2] to analyse the load combination for the assumption of wind turbine in operation may be too strict. When such an unlikely situation is encountered, the fulfilment of this requirement of [4.6.7.2] may be deviated from in the following manner: The load combination under investigation shall still be analysed for the assumption of wind turbine in operation; however, the requirements to the return periods of the wave, ice, current and water level conditions that the wind load effect is combined with may be relaxed and set lower than the values specified in Table 4-9 for the particular load combination, as long as it can be documented that the return period for the resulting combined load effect does not fall below 50 years.

**Guidance note:**

When the fetch is limited and wind and waves have the same direction in severe storms, then the wind climate intensity is likely to reach its extreme maximum at the same time as the wave climate intensity reaches its extreme maximum, and it may be unlikely to see wind speeds below the cut-out wind speed during the presence of the 50-year wave climate. Likewise, it may be unlikely to see the 50-year wave climate during operation of the wind turbine. When the topography, e.g. in terms of a nearby coastline, forces the extreme maximum of the wind climate to take place at a different time than the extreme maximum of the wave climate intensity, then it may be likely to see wind speeds below the cut-out wind speed during the presence of the 50-year wave climate. Likewise, it may be likely to see the 50-year wave climate during operation of the wind turbine.

When a large fetch is present, there may be a phase difference between the occurrence of the extreme maximum of the wind climate intensity and the extreme maximum of the wave climate intensity, and it may be likely to see wind speeds below the cut-out wind speed during the presence of the 50-year wave climate. Likewise, it may be likely to see the 50-year wave climate during operation of the wind turbine.

4.6.7.4 Every time a load case is investigated, which contains a load effect contribution from ice loads, loads from moving ice shall be considered as well as loads from fast-frozen ice and loads due to temperature fluctuations in the ice.

4.6.7.5 Load combination No. 5 in Table 4-9 is of relevance for structures in waters which are covered by ice every year. Investigations for Load combination No. 5 in Table 4-9 can be waived for structures in waters which are covered by ice less frequently than every year.

4.6.7.6 When a load case is investigated, which contains a load effect contribution from wave loads, loads from wave trains in less severe sea states than the sea state of the specified return period shall be considered if these loads prove to produce a larger load effect than the sea state of the specified return period.

**Guidance note:**

Dynamic effects may cause less severe sea states than the sea state of the specified return period to produce more severe load effects, e.g. if these sea states imply wave trains arriving at the wind turbine structure with frequencies which coincide with a frequency of one of the natural vibration modes of the structure. The possibility that waves break at the wind turbine structure may play a role in this context and should be included in the considerations.

4.6.7.7 Co-directionality of wind and waves may be assumed for calculation of the wave loads acting on the support structure for all design cases except those corresponding to the wind turbine in a parked (standstill or idling) design situation. The misalignment of wind and wave directions in the parked situation is to be accounted for.

**Guidance note:**

Allowance for short term deviations from the mean wind direction in the parked situation should be made by assuming a constant yaw misalignment. It is recommended to apply a yaw misalignment of ±15°.

In areas where swell may be expected, special attention needs to be given to swell, which has a low correlation with wind speed and wind direction.

4.6.7.8 The multi-directionality of the wind and the waves may in some cases have an important influence on the loads acting on the support structure, depending primarily on whether the structure is axisymmetric or not. For some design load cases the load calculations may be undertaken by assuming that the wind and the waves...
are acting co-directionally from a single, worst case direction.

4.6.7.9 Characteristic extreme wind load effects are in this standard defined as wind load effects with a 50-year return period. 5-year wind load effects form part of some load combinations. When only wind load effects with a 100-year return period are available, the 100-year wind load effects have to be converted to 50-year values. This can be done by multiplication by a conversion factor. Likewise, to the extent that 5-year wind load effects are needed in load combinations and only 50-year values are available, the 50-year values have to be converted to 5-year values for use in these load combinations.

Guidance note:
The ratio \( \frac{F_{\text{wind,100}}}{F_{\text{wind,50}}} \) between the 100- and 50-year wind load effects depends on the coefficient of variation in the distribution of the annual maximum wind load and can be used as a conversion factor to achieve the 50-year wind load effect \( F_{\text{wind,50}} \) in cases where only the 100-year value \( F_{\text{wind,100}} \) is available. Unless data indicate otherwise, the ratio \( \frac{F_{\text{wind,100}}}{F_{\text{wind,50}}} \) can be taken from Table 4-10. Table 4-10 also gives the ratio \( \frac{F_{\text{wind,5}}}{F_{\text{wind,50}}} \) between the 5-year wind load effect \( F_{\text{wind,5}} \) and the 50-year wind load effect \( F_{\text{wind,50}} \). This is useful in some load combinations that require the 5-year wind load effect. Table 4-10 is based on an assumption of a Gumbel-distributed annual maximum wind load effect.

<table>
<thead>
<tr>
<th>Coefficient of variation of annual maximum wind load effect (%)</th>
<th>Ratio between 100- and 50-year wind load effects, ( \frac{F_{\text{wind,100}}}{F_{\text{wind,50}}} )</th>
<th>Ratio between 5- and 50-year wind load effects, ( \frac{F_{\text{wind,5}}}{F_{\text{wind,50}}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>1.05</td>
<td>0.85</td>
</tr>
<tr>
<td>15</td>
<td>1.06</td>
<td>0.80</td>
</tr>
<tr>
<td>20</td>
<td>1.07</td>
<td>0.75</td>
</tr>
<tr>
<td>25</td>
<td>1.08</td>
<td>0.72</td>
</tr>
<tr>
<td>30</td>
<td>1.09</td>
<td>0.68</td>
</tr>
<tr>
<td>35</td>
<td>1.10</td>
<td>0.64</td>
</tr>
</tbody>
</table>

The conversion factors are given as functions of the coefficient of variation of the annual maximum wind load effect. There is no requirement in this standard to document this coefficient of variation. Note that use of the conversion factor \( \frac{F_{\text{wind,5}}}{F_{\text{wind,50}}} \) given in Table 4-10 to obtain the 5-year wind load effect from the 50-year wind load effect will be nonconservative if the distribution of the annual maximum wind load effect is not a Gumbel distribution and has a less heavy upper tail than the Gumbel distribution. Note also that for a particular wind turbine, the coefficient of variation of the annual maximum wind load effect may be different depending on whether the wind turbine is located on an offshore location or on an onshore location. For offshore wind turbines the coefficient of variation is assumed to have a value of approximately 20 to 30%.

4.6.8 Transient load cases

4.6.8.1 Actuation loads from the operation and control of the wind turbine produce transient wind loads on the wind turbine structure. The following events produce transient loads and shall be considered:

— start up from stand-still or from idling
— normal shutdown
— emergency shutdown
— normal fault events: faults in control system and loss of electrical network connection
— abnormal fault events: faults in protection system and electrical systems
— yawing.

4.6.8.2 The characteristic transient wind load effect shall be calculated as the maximum load effect during a 10-minute period whose wind intensity shall be taken as the most unfavourable 10-minute mean wind speed in the range between the cut-in wind speed and the cut-out wind speed. In order to establish the most critical wind speed, i.e., the wind speed that produce the most severe load during the transient loading, gusts, turbulence, shift in wind direction, wind shear, timing of fault situations, and grid loss in connection with deterministic gusts shall be considered.

4.6.8.3 The characteristic transient wind load effect shall be combined with the 10-year wave load effect. The combination may be worked out according to the linear combination format to produce the design load effect from the separately calculated characteristic wind load effect and wave load effect. The combination may alternatively be worked out by direct simulation of the characteristic combined load effect in a structural analysis in the time domain for simultaneously applied simulated time series of the wind load and the wave load.

4.6.8.4 When transient wind loads are combined with wave loads, misalignment between wind and waves shall be considered. For non-axisymmetric support structures, the most unfavourable wind load direction and wave load direction shall be assumed.
4.6.9 Load combination for the fatigue limit state

For analyses of the fatigue limit state, a characteristic load effect distribution shall be applied which is the expected load effect distribution over the design life. The expected load effect distribution is a distribution of stress ranges owing to load fluctuations and contains contributions from wind, waves, current, ice and water level as well as from possible other sources. The expected load effect distribution shall include contributions from

— wind turbine in operation
— parked wind turbine (idling and standing still)
— start up
— normal shutdown
— control, protection and system faults, including loss of electrical network connection
— transport and assembly prior to putting the wind turbine to service
— maintenance and repair during the service life.

For fatigue analysis of a foundation pile, the characteristic load effect distribution shall include the history of stress ranges associated with the driving of the pile prior to installing the wind turbine and putting it to service.

Guidance note:
The characteristic load effect distribution can be represented as a histogram of stress ranges, i.e., the number of constant-range stress cycles is given for each stress range in a sufficiently fine discretisation of the stress ranges. The individual contributions to this load effect distribution from different sources can be represented the same way.

For contributions to the expected load effect distribution that are consecutive in time or otherwise mutually exclusive, such as the contribution from the transportation and installation phase and the contribution from the in-service phase, the fatigue damage due to each contribution can be calculated separately and added together without introducing any particular prior combination of the contributions to the distribution. Alternatively, the different contributions can be combined to form the expected load effect distribution prior to the fatigue damage calculation by adding together the number of stress cycles at each defined discrete stress range from the respective underlying distributions.

When the expected load effect distribution contains load effects which result from two or more concurrently acting load processes, such as a wind load and a concurrent wave load, the respective underlying stress range distributions for separate wind load effect and separate wave load effect need to be adequately combined prior to the calculation of the fatigue damage. When wind loads and wave loads act concurrently, it can be expected that their combined load effect distribution will contain somewhat higher stress ranges than those of the underlying individual wave load effect and wind load effect distributions. The following idealised approach to combination of the two underlying stress range distributions will usually be conservative: The number of stress cycles of the combined load effect distribution is assumed equal to the number of stress cycles in that of the underlying distributions (i.e. the distribution of wind stress cycles and the distribution of wave stress cycles) which contains the highest number of cycles. Then the largest stress range in the wind load effect distribution is combined by the largest stress range in the wave load effect distribution by simple superposition, the second largest stress ranges are combined analogously, the third largest stress ranges the same, and so on.

There may be some ambiguity involved with how concurrent wave load effects and wind load effects shall be combined to form the resulting load effect distribution for fatigue damage prediction. The proposed method of combination is idealised and implies an assumption of colinear wind and waves. However, when combining wind load effects and wave load effects for fatigue, consideration of the distribution of the wind direction, the distribution of the wave direction and the distribution of the misalignment between wind and waves is important and may, relative to the situation with colinear wind and waves, often imply gains in terms of reduced fatigue damage that will more than outweigh the possible effects of conservatism in the idealised combination method. Caution should be exercised when counting on such gains when wind and waves are not colinear, since situations may exist for which larger fatigue damage will accumulate if the waves act perpendicularly to the wind rather than colinearly with the wind.

4.7 Load effect analysis

4.7.1 General

4.7.1.1 Load effects, in terms of motions, displacements, and internal forces and stresses in the wind turbine 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.
4.7.1.2 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.

4.7.1.3 In general, three frequency bands need to be considered for offshore structures:

<table>
<thead>
<tr>
<th>Frequency Band</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>High frequency (HF)</td>
<td>Rigid body natural periods below the dominating wave periods, e.g. ringing and springing responses.</td>
</tr>
<tr>
<td>Wave frequency (WF)</td>
<td>Typically wave periods in the range 4 to 25 seconds. Applicable to all offshore structures located in the wave active zone.</td>
</tr>
<tr>
<td>Low frequency (LF)</td>
<td>This frequency band relates to slowly varying responses with natural periods beyond those of the dominating wave energy (typically slowly varying motions).</td>
</tr>
</tbody>
</table>

4.7.1.4 For fully restrained structures a static or dynamic wind-wave-structure-foundation analysis is required.

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

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

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

4.7.1.8 Earthquake loads need only be considered for restrained modes of behaviour.

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

4.7.3 Load effects in structures and foundation soils
4.7.3.1 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.

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

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

4.8 Accidental loads

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

— dropped objects
— collision impact
— explosions
— fire
— change of intended pressure difference
— load from rare, large breaking wave
— accidental impact from vessel, helicopter or other objects.

4.8.1.2 Relevant accidental loads should be determined on the basis of an assessment and relevant experiences. Accidental loads from ship impacts are given in [4.4.3].

4.8.1.3 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.
4.9 Deformation loads

4.9.1 General
Deformation loads are loads caused by inflicted deformations such as:
— temperature loads
— built-in deformations
— settlement of foundations.

4.9.2 Temperature loads
4.9.2.1 Structures shall be designed for the most extreme temperature differences they may be exposed to.
4.9.2.2 The ambient sea or air temperature shall be calculated as the extreme value whose return period is 50 years.
4.9.2.3 Structures shall be designed for a solar radiation intensity of 1000 W/m².

4.9.3 Settlements
Settlement of the support structure and its foundation due to vertical deformations of the supporting soils shall be considered. This includes consideration of differential settlements.
SECTION 5 LOAD AND RESISTANCE FACTORS

5.1 Load factors

5.1.1 Load factors for the ULS

5.1.1.1 Table 5-1 provides three sets of load factors to be used when characteristic loads or load effects from different load categories are combined to form the design load or the design load effect for use in design. For analysis of the ULS, the sets denoted (a) and (b) shall be used when the characteristic environmental load or load effect is established as the 98% quantile in the distribution of the annual maximum load or load effect. For analyses of the ULS for abnormal wind load cases, the set denoted (c) shall be used.

The load factors apply in the operational condition as well as in the temporary condition. The load factors are generally applicable for all types of support structures and foundations and they apply to design of support structures and foundations which qualify for design to the normal safety class.

Guidance note:
For global analysis and design of primary structures, load factor set (b) is often more critical than load factor set (a). However, there are cases for which load factor set (a) is more critical than load factor set (b), such as – but not limited to – design against lifting forces, hydrostatic pressures and ship impacts. It is therefore important that both load factor sets are always checked as required.

Load factor set (a) is in particular of relevance for design of secondary structures such as boat landings, fenders and lay down areas, for which variable functional loads from ship impacts are the dominating loads.

There are usually weather restrictions in place for when service vessels can be operated. To the extent that it is necessary to combine a functional ship impact load from a service vessel with an environmental load according to load factor set (a), the characteristic environmental load will have to be taken as the 98% quantile in the distribution of the annual maximum environmental load conditioned on the environment being below the specified threshold for operation of the service vessels. This is a load which will be smaller than the characteristic environmental load for the situation that no weather restrictions are in place. The distribution of the annual maximum environmental load conditioned on the environment being below the specified threshold for operation of the service vessels comes about by a truncation of the upper tail of the distribution of the annual maximum environmental load for the situation that no weather restrictions are in place.

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

5.1.1.2 The characteristic environmental load effect (E), which forms part of the load combinations of Table 5-1, is to be taken as the characteristic combined load effect, determined according to Sec.4, and representing the load effect that results from two or more concurrently acting load processes.

5.1.1.3 For permanent loads (G) and variable functional loads (Q), the load factor in the ULS shall normally be taken as $\psi = 1.0$ for load combinations (b) and (c).

5.1.1.4 When a permanent load (G) or a variable functional load (Q) is a favourable load, then a load factor $\psi = 0.9$ shall be applied for this load in combinations (b) and (c) of Table 5-1 instead of the value of 1.0 otherwise required. The only exception from this applies to favourable loads from foundation soils in geotechnical engineering problems, for which a load factor $\psi = 1.0$ shall be applied. A load is a favourable load when a reduced value of the load leads to an increased load effect in the structure.

Guidance note:
One example of a favourable load is the weight of a soil volume which has a stabilising effect in an overturning problem for a foundation.
Another example consists of gravity loads that significantly relieve the total load response.

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

5.1.1.5 For design to high safety class, the requirements to the load factor $\gamma$ specified for design to normal safety class in Table 5-1 for environmental loads shall be increased by 13%.

5.1.2 Load factor for the FLS

5.1.2.1 The structure shall be able to resist expected fatigue loads, which may occur during temporary and operational design conditions. Whenever significant cyclic loads may occur in other phases, e.g. during manufacturing and transportation, such cyclic loads shall be included in the fatigue load estimates.

5.1.2.2 The load factor $\gamma$ in the FLS is 1.0 for all load categories.

5.1.3 Load factor for the SLS

For analysis of the SLS, the load factor $\gamma$ is 1.0 for all load categories, both for temporary and operational design conditions.

5.1.4 Load factor for the ALS

The load factor $\gamma$ for the ALS is 1.0.

5.2 Resistance factors

5.2.1 Resistance factors for the ULS

5.2.1.1 Resistance factors for the ULS are given in the relevant sections for design in the ULS. These resistance factors apply to design of support structures and foundations which qualify for design to normal safety class.

5.2.1.2 For design of support structures and foundations to high safety class, the same resistance factors as those required for design to normal safety class can be applied, provided the load factors for environmental loads are taken in accordance with [5.1.1.5].

5.2.2 Resistance factors for the FLS

Resistance factors for the FLS are given in the relevant sections for design in the FLS.

5.2.3 Resistance factors for the ALS and the SLS

Unless otherwise stated, the material factor $\gamma_m$ for the ALS and the SLS shall be taken as 1.0.
SECTION 6 MATERIALS

6.1 Selection of steel materials and inspection principles

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

6.1.2 Design temperatures

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

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

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

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

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

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

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

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

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

6.1.3 Structural category

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

Guidance note:
Conditions that may result in brittle fracture are to be avoided. Brittle fracture may occur under a combination of:
- presence of sharp defects such as cracks
- high tensile stress in direction normal to planar defect(s)
- material with low fracture toughness.
Sharp cracks resulting from fabrication may be found by inspection and repaired. Fatigue cracks may also be discovered during service life by inspection.
High stresses in a component may occur due to welding. A complex connection is likely to provide more restraint and larger residual stress than a simple one. This residual stress may be partly removed by post weld heat treatment if necessary. Also a complex connection shows a more three-dimensional stress state due to external loading than simple connections. This stress state may provide basis for a cleavage fracture.
The fracture toughness is dependent on temperature and material thickness. These parameters are accounted for separately in selection of material. The resulting fracture toughness in the weld and the heat affected zone is also dependent on the fabrication method.
Thus, to avoid brittle fracture, first a material with suitable fracture toughness for the actual design temperature and thickness is selected. Then a proper fabrication method is used. In special cases post weld heat treatment may be performed to reduce crack driving stresses, see also DNV-OS-C401. A suitable amount of inspection is carried out to remove planar defects larger than those considered acceptable. In this standard selection of material with appropriate fracture toughness and avoidance of unacceptable defects are achieved by linking different types of connections to different structural categories and inspection categories.
6.1.3.2 Components are classified into structural categories according to the following criteria:

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

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

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6.1.3.3 The structural category for selection of materials shall be determined according to principles given in Table 6-1.

<table>
<thead>
<tr>
<th>Structural category</th>
<th>Principles for determination of structural category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special</td>
<td>Structural parts where failure will have substantial consequences and are subject to a stress condition that may increase the probability of a brittle fracture. ¹)</td>
</tr>
<tr>
<td>Primary</td>
<td>Structural parts where failure will have substantial consequences.</td>
</tr>
<tr>
<td>Secondary</td>
<td>Structural parts where failure will be without significant consequence.</td>
</tr>
</tbody>
</table>

¹) In complex joints a triaxial or biaxial stress pattern will be present. This may give conditions for brittle fracture where tensile stresses are present in addition to presence of defects and material with low fracture toughness.

Guidance note:
Monopile structures are categorised as “Primary”, because they are non-redundant structures whose stress pattern is primarily uniaxial and whose risk of brittle fracture is negligible.

Likewise, towers are also categorised as “Primary”.

Tubular joints are categorised as “Special” due to their biaxial or triaxial stress patterns and risk of brittle fracture. This will influence the thickness limitations as specified in Table 6-7.

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6.1.3.4 Requirements and guidance for manufacturing of steel materials are given in DNV-OS-C401. For supplementary guidance, reference is made to EN 1090-1 and ENV 1090-5. 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 6-2 shall be presented.

<table>
<thead>
<tr>
<th>Certification process</th>
<th>Material certificate (EN10204)</th>
<th>Structural category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test certificate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>As work certificate, inspection and tests witnessed and signed by an independent third party body</td>
<td>3.2 Special</td>
<td></td>
</tr>
<tr>
<td>Work certificate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test results of all specified tests from samples taken from the products supplied. Inspection and tests witnessed and signed by QA department</td>
<td>3.1 Primary</td>
<td></td>
</tr>
<tr>
<td>Test report</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Confirmation by the manufacturer that the supplied products fulfil the purchase specification, and test data from regular production, not necessarily from products supplied</td>
<td>2.2 Secondary</td>
<td></td>
</tr>
</tbody>
</table>

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

6.1.3.6 The inspection category is by default related to the structural category according to Table 6-3.

<table>
<thead>
<tr>
<th>Inspection category</th>
<th>Structural category</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Special</td>
</tr>
<tr>
<td>II</td>
<td>Primary</td>
</tr>
<tr>
<td>III</td>
<td>Secondary</td>
</tr>
</tbody>
</table>
6.1.3.7 The weld connection between two components shall be assigned an inspection category according to the highest category of the joined components. For stiffened plates, the weld connection between stiffener and stringer and girder web to the plate may be inspected according to inspection Category III.

6.1.3.8 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 required for elements within structural category primary may be reduced, but the extent must not be less than that for inspection Category III.

6.1.3.9 Fatigue-critical details within structural category primary and secondary shall be inspected according to requirements given for Category I. This requirement applies to fatigue-critical details in the support structure and the foundation, but not in the tower.

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

6.1.3.11 For monopile type structures, the longitudinal welds in the monopile and in the transition piece to the grouted connection shall be inspected according to requirements given for Category I.

6.1.4 Structural steel

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

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

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

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

6.1.4.5 In structural cross-joints where high tensile stresses are acting perpendicular to the plane of the plate, the plate material shall be tested to prove the ability to resist lamellar tearing, Z-quality, see [6.1.4.11].

6.1.4.6 Requirements for forgings and castings are given in DNV-OS-B101.

6.1.4.7 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 DNV-OS-B101 Ch.2 Sec.1. The steel grades are referred to as NV grades. Structural steel designations for various strength groups are referred to as given in Table 6-4.

<table>
<thead>
<tr>
<th>Table 6-4 Material designations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Designation</td>
</tr>
<tr>
<td>-------------</td>
</tr>
<tr>
<td>NV</td>
</tr>
<tr>
<td>NV-27</td>
</tr>
<tr>
<td>NV-32</td>
</tr>
<tr>
<td>NV-36</td>
</tr>
<tr>
<td>NV-40</td>
</tr>
<tr>
<td>NV-420</td>
</tr>
<tr>
<td>NV-460</td>
</tr>
<tr>
<td>NV-500</td>
</tr>
<tr>
<td>NV-550</td>
</tr>
<tr>
<td>NV-620</td>
</tr>
<tr>
<td>NV-690</td>
</tr>
</tbody>
</table>

¹ For steels of improved weldability the required specified minimum yield stress is reduced for increasing material thickness, see DNV-OS-B101.

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

— steels of normal weldability
— steels of improved weldability.
The two series are intended for the same applications. However, the improved weldability grades have, in addition to leaner chemistry and better weldability, extra margins to account for reduced toughness after welding. These grades are also limited to a specified minimum yield stress of 500 N/mm².

6.1.4.9 Conversions between NV grades as used in Table 6-4 and steel grades used in the EN 10025-2, -3, -4 and -6 standards are used for the sole purpose of determining plate thicknesses and are given in Table 6-5.

<table>
<thead>
<tr>
<th>NV grade</th>
<th>EN 10025-2</th>
<th>EN 10025-3</th>
<th>EN 10025-4</th>
<th>EN 10025-6</th>
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</tr>
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<td>NV B</td>
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<td>–</td>
</tr>
<tr>
<td>NV C</td>
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<td>–</td>
<td>–</td>
</tr>
<tr>
<td>NV E</td>
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<td>–</td>
<td>–</td>
<td>–</td>
</tr>
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</tr>
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<td>S420ML</td>
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<td>(S420ML)</td>
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</tr>
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<td>S460ML, (S460M)</td>
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<td>S460ML</td>
<td>S460QL</td>
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<td>(S460ML)</td>
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<td>(S500QL1)</td>
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<td>S550QL</td>
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<tr>
<td>NV F690</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

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

Guidance note:
Important notes to the conversions between NV grades and EN grades in Table 6-5:
The conversions are based on comparable requirements for strength and toughness. NV grades are, in general, better steel qualities than comparable EN 10025-2 grades. For example, all NV grades except NV A and NV B, are fully killed and fine grain treated. This is the case only for the J2G3 and K2G3 grades in EN 10025-2.

The delivery condition is specified as a function of thickness for all NV 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 standard. In EN 10025-2, minimum specified mechanical properties (yield stress, tensile strength range and elongation) are thickness dependent. The corresponding properties for NV grades are specified independently of thickness. 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 NV grades.

6.1.4.10 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 6-6.

Additional symbol:

\[ Z = \text{steel grade of proven through-thickness properties. This symbol is omitted for steels of improved weldability although improved through-thickness properties are required.} \]

<table>
<thead>
<tr>
<th>Strength group</th>
<th>Grade</th>
<th>Normal weldability</th>
<th>Improved weldability</th>
<th>Test temperature (°C)</th>
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</tr>
<tr>
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<td></td>
</tr>
<tr>
<td></td>
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<td>AHW</td>
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</tr>
<tr>
<td></td>
<td>DH</td>
<td>DHW</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>EH</td>
<td>EHW</td>
<td>–40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FH</td>
<td>–</td>
<td>–60</td>
<td></td>
</tr>
<tr>
<td>EHS</td>
<td>AEH</td>
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<td>0</td>
<td></td>
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<td>DEHW</td>
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<td>EEH</td>
<td>EEHW</td>
<td>–40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FEH</td>
<td>–</td>
<td>–60</td>
<td></td>
</tr>
</tbody>
</table>

1) Charpy V-notch tests are required for thickness above 25 mm but is subject to agreement between the contracting parties for thickness of 25 mm or less.

6.1.4.11 The grade of steel to be used shall in general be selected according to the design temperature and the thickness for the applicable structural category as specified in Table 6-7. The steel grades in Table 6-7 are NV grade designations.

6.1.4.12 For the case that the required grade of steel to be used comes out as NV grade F according to Table 6-7, the requirement for the grade of steel can be relaxed to NV grade E when the design temperature is \(-20°C\) or higher. In this case, the test temperature can be increased from \(-60°C\) to \(-40°C\) according to Table 6-6.
6.1.4.13 Selection of a better steel grade than minimum required in design shall not lead to more stringent requirements in fabrication.

6.1.4.14 The grade of steel to be used for thickness less than 10 mm and/or design temperature above 0°C may be specially considered in each case. For submerged structures, i.e. for structures below LAT−1.5 m, such as in the North Sea, the design temperature will be somewhat above 0°C (typically 2°C) and special considerations can be made in such cases.

6.1.4.15 Welded steel plates and sections of thickness exceeding the upper limits for the actual steel grade as given in Table 6-7 shall be evaluated in each individual case with respect to the fitness for purpose of the weldments. The evaluation should be based on fracture mechanics testing and analysis, e.g. in accordance with BS 7910.

6.1.4.16 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 6-7.

6.1.4.17 The use of steels with specified minimum yield stress greater than 550 N/mm² (NV550) shall be subject to special consideration for applications where anaerobic environmental conditions such as stagnant water, organically active mud (bacteria) and hydrogen sulphide may predominate.

6.1.4.18 Predominantly anaerobic conditions can for this purpose be characterised by a concentration of sulphate reducing bacteria, SRB, in the order of magnitude < 10³ SRB/ml, determined by method according to NACE TPC Publication No. 3.

6.1.4.19 The susceptibility of the steel to hydrogen-induced stress cracking (HISC) shall be specially considered when used for critical applications. See also Sec.11.

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

Table 6-7 Thickness limitations (mm) of structural steels for different structural categories and design temperatures (°C)

<table>
<thead>
<tr>
<th>Structural Category</th>
<th>Grade</th>
<th>≥10</th>
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<th>−10</th>
<th>−20</th>
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<td>Secondary</td>
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<td>25</td>
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<tr>
<td></td>
<td>B/BW</td>
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<td>60</td>
<td>50</td>
<td>40</td>
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<tr>
<td></td>
<td>D/DW</td>
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<td>150</td>
<td>100</td>
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</tr>
<tr>
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<td>E/EW</td>
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<td>150</td>
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<tr>
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<td>DH/DHW</td>
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<td>100</td>
<td>80</td>
<td>60</td>
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<td>150</td>
<td>150</td>
<td>150</td>
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N.A. = no application
6.1.5 Post weld heat treatment (PWHT)

Except for joints in monopiles, 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 DNV-OS-C401 Ch.2 Sec.2 F200. 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.

6.2 Selection of concrete materials

6.2.1 General

6.2.1.1 For selection of structural concrete materials, DNV-OS-C502 Sec.4, “Materials” shall apply.

6.2.1.2 This subsection for selection of concrete materials provides a short summary of DNV-OS-C502 Sec.4, focusing on issues which typically pertain to offshore concrete structures, but not necessarily to standard concrete design. For all design purposes, the user should always refer to the complete description in DNV-OS-C502, and the text in this subsection shall be considered application text for the text of DNV-OS-C502 with respect to offshore wind turbine concrete structures.

6.2.2 Material requirements

6.2.2.1 The materials selected for the load-bearing structures shall be suitable for the purpose. The material properties and verification that these materials fulfil the requirements shall be documented.

6.2.2.2 The materials, all structural components and the structure itself shall be ensured to maintain the specified quality during all stages of construction and for the intended structural life.

6.2.2.3 Constituent materials for structural concrete are cement, aggregates and water. Structural concrete may also include admixtures and additions.

6.2.2.4 Constituent materials shall be sound, durable, free from defects and suitable for making concrete that will attain and retain the required properties. Constituent materials shall not contain harmful ingredients in quantities that can be detrimental to the durability of the concrete or cause corrosion of the reinforcement and shall be suitable for the intended use.

6.2.2.5 The following types of Portland cement are, in general, assumed to be suitable for use in structural concrete and/or grout in a marine environment if unmixed with other cements:

— Portland cements
— Portland composite cements
— Blastfurnace cements, with high clinker content.

Provided suitability is demonstrated also the following types of cement may be considered:

— Blastfurnace cements
— Pozzolanic cements
— Composite cements.

The above types of cement have characteristics specified in international and national standards. They can be specified in grades based on the 28-day strength in mortar. Cements shall normally be classified as normal hardening, rapid hardening or slowly hardening cements.

**Guidance note:**
Low heat cement may be used where heat of hydration may have an adverse effect on the concrete during curing.

6.2.2.6 The required water content is to be determined by considering the strength and durability of hardened concrete and the workability of fresh concrete. The water-to-cement ratio by weight may be used as a measure. For requirements to the water-to-cement ratio, see [6.2.3.5].

6.2.2.7 Salt water, such as raw seawater, shall not be used as mixing or curing water for structural concrete.

6.2.2.8 Normal weight aggregates shall, in general, be of natural mineral substances. They shall be either crushed or uncrushed with particle sizes, grading and shapes such that they are suitable for the production of concrete. Relevant properties of aggregate shall be defined, e.g. type of material, shape, surface texture, physical properties and chemical properties.

Aggregates shall be free from harmful substances in quantities that can affect the properties and the durability of the concrete adversely. Examples of harmful substances are claylike and silty particles, organic materials and sulphates and other salts.
6.2.2.9 Aggregates shall be evaluated for risk of Alkali Silica Reaction (ASR) in concrete according to internationally recognised test methods. Suspect aggregates shall not be used unless specifically tested and approved. The approval of aggregates that might combine with the hydration products of the cement to cause ASR shall state which cement the approval applies to. The aggregate for structural concrete shall have sufficient strength and durability.

6.2.2.10 An appropriate grading of the fine and coarse aggregates for use in concrete shall be established. The grading and shape characteristics of the aggregates shall be consistent throughout the concrete production.

6.2.2.11 Maximum aggregate size is to be specified based on considerations concerning concrete properties, spacing of reinforcement and cover to the reinforcement.

6.2.2.12 Latent hydraulic or pozzolanic supplementary materials such as silica fume, pulverized fly ash and granulated blast furnace slag may be used as additions. The amount is dependent on requirements to workability of fresh concrete and required properties of the hardened concrete. The content of silica fume used as additions should normally not exceed 10% of the weight of Portland cement clinker. When fly ash, slag or other pozzolana is used as additions, their content should normally not exceed 35% of the total weight of cement and additions. When Portland cement is used in combination with only ground granulated blast furnace slag, the slag content may be increased. The clinker content shall, however, not be less than 30% of the total weight of cement and slag.

6.2.2.13 The composition and properties of repair materials shall be such that the material fulfils its intended use. Only materials with established suitability shall be used. Emphasis shall be given to ensure that such materials are compatible with the adjacent material, particularly with regard to the elasticity and temperature dependent properties.

6.2.3 Concrete

6.2.3.1 Normal Strength Concrete is a concrete of grade C35 to C65.

6.2.3.2 High Strength Concrete is a concrete of grade in excess of C65.

6.2.3.3 The concrete composition and the constituent materials shall be selected to satisfy the requirements of DNV-OS-C502 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.

6.2.3.4 The required properties of fresh and hardened concrete shall be specified. These required properties shall be verified by the use of recognised testing methods, international standards or recognised national standards. Recognised standard is relevant ASTM, ACI and EN standard.

6.2.3.5 Compressive strength shall always be specified. In addition, tensile strength, modulus of elasticity (E-modulus) and fracture energy may be specified. Properties which can cause cracking of structural concrete shall be accounted for, i.e. creep, shrinkage, heat of hydration, thermal expansion and similar effects. The durability of structural concrete is related to permeability, absorption, diffusion and resistance to physical and chemical attacks in the given environment, a low water/cement-binder ratio is generally required in order to obtain adequate durability. The concrete shall normally have a water/cement-binder ratio not greater than 0.45. In the splash zone, this ratio shall not be higher than 0.40.

6.2.3.6 The demands given for cement content in DNV-OS-C502 Sec.4 C209 shall be considered as demands for cement/filler content calculated according to a recognised standard. The demands may be waived based on conditions such as less strict national requirements or track records for good performance and durability in marine environments for similar structures.

6.2.3.7 The concrete grades are defined as specified in DNV-OS-C502 Sec.4 Tables C1 and C2 as a function of the Characteristic Compressive Cylinder strength of the concrete, $f_{ck}$. The properties of hardened concrete are in general related to the concrete grade. For concrete exposed to seawater the minimum grade is C40. Further limitations to minimum concrete grades are given in DNV-OS-C502, Sec.4C.

6.2.4 Grout and mortar

6.2.4.1 The mix design of grout and mortar shall be specified for its designated purpose.

6.2.4.2 The constituents of grout and mortar shall meet the same type of requirements for their properties as those given for the constituents of concrete.

6.2.5 Reinforcement steel

6.2.5.1 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.

6.2.5.2 Reinforcement steel shall comply with ISO 6935, Parts 2 and 3 or relevant national or international standards for reinforcement steel.

6.2.5.3 Consistency shall be ensured between material properties assumed in the design and the requirements of the standard used. In general, hot-rolled, ribbed bars of weldable quality and with high ductility shall be used. Where the use of seismic detailing is required, the reinforcement provided shall meet the ductility requirements of the reference standard used in the design.

6.2.5.4 Fatigue properties and S-N curves shall be consistent with the assumptions of design.

6.2.6 Prestressing steel

Prestressing steel shall comply with ISO 6934 and/or relevant national or international standards for prestressing steel.

6.3 Grout materials and material testing

6.3.1 General

6.3.1.1 The grout materials for grouted connections shall comply with relevant requirements given for grout in DNV-OS-C502 “Offshore Concrete Structures”, Sec.4. Structural grout materials shall be certified in accordance with the requirements in DNV-OS-C502, Sec.9E.

6.3.1.2 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; reference is made to DNV-OS-C502, Appendix H.

6.3.1.3 In general there are two types of structural grouts used for offshore wind turbine structures; neat cement grout (typical for jacket structures) and pre-packed blended grout (typical for monopile structures). Each grout type may require different test programs depending on the relevant material properties and application. For pre-packed blended grouts the material testing, relevant test methods and requirements for testing are outlined in DNV-OS-C502, Appendix H. For neat cement grout the extent and method of testing shall be agreed in advance with the classification society.

6.3.1.4 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 structure, it shall be confirmed that the maximum temperature rise caused by the hardening process is within acceptable limits.

6.3.1.5 Samples for quality control during offshore grouting of neat cement grout or pre-packed blended 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 are normally taken at the mixer.

6.3.1.6 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. If the grout is to 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 verified.

6.3.1.7 The required test methods and testing frequency for offshore quality control testing is stated in DNV-OS-C502. Sec.7F.

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.
7.1 Ultimate limit states – general

7.1.1 General

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

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

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

7.1.1.4 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.

7.1.1.5 Prediction of structural capacity shall be carried out with due consideration of capacity reductions which are implied by the corrosion allowance specified in Sec.11.

The increase in wall thickness for a structural component, added to allow for corrosion, shall not be included in the calculation of the structural capacity of the component.

Guidance note:
Structural design in the ULS can be based on a steel wall thickness equal to the nominal thickness reduced by the corrosion allowance over the full service life, in which 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 and (2) 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 in [11.2.2].

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.

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.

7.1.2 Structural analysis

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

7.1.2.2 When plastic or elastic-plastic analyses are used for structures exposed to cyclic loading, i.e. 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. 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.

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

7.1.2.4 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.

7.1.2.5 When plastic analysis and/or plastic capacity checks are used (cross section type I and II, according to App.H), 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.

7.1.2.6 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 Appendix H.

7.1.3 Ductility

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

7.1.3.2 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.

7.1.4 Yield check

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

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

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

7.1.4.4 For yield check of welded connections, see [7.8] regarding welded connections.

7.1.5 Buckling check

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

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

7.1.5.3 The characteristic buckling strength shall be based on the 5th percentile of test results.

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

7.1.5.5 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.

7.1.6 Vibrations

Vibrations in secondary structures such as internal and external J-tubes are undesirable. An assessment of vibrations in J-tubes shall be performed, either based on experience from similar structures or by calculations.

Guidance note:
Vibrations in J-tubes have occurred in the past and therefore this issue needs to be properly considered in new designs. For designs involving free-hanging cables, long-term impacts from vibrations should be considered.

7.2 Ultimate limit states – shell structures

7.2.1 General

7.2.1.1 The buckling stability of shell structures may be checked according to DNV-RP-C202 or Eurocode 3/EN 1993-1-1 and EN 1993-1-6.

7.2.1.2 For interaction between shell buckling and column buckling, DNV-RP-C202 may be used.
7.2.1.3 If DNV-RP-C202 is applied, the material factor for shells shall be in accordance with Table 7-1.

<table>
<thead>
<tr>
<th>Type of structure</th>
<th>$\lambda \leq 0.5$</th>
<th>$0.5 &lt; \lambda &lt; 1.0$</th>
<th>$\lambda \geq 1.0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder, beams stiffeners on shells</td>
<td>1.10</td>
<td>1.10</td>
<td>1.10</td>
</tr>
<tr>
<td>Shells of single curvature (cylindrical shells, conical shells)</td>
<td>1.10</td>
<td>$0.80 + 0.60 \lambda$</td>
<td>1.40</td>
</tr>
</tbody>
</table>

**Guidance note:**
Note that the slenderness is based on the buckling mode under consideration.

\[
\lambda = \frac{f_y}{\sigma_e}
\]

- \( \lambda \) = reduced slenderness parameter
- \( f_y \) = specified minimum yield stress
- \( \sigma_e \) = elastic buckling stress for the buckling mode under consideration

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

7.3 Ultimate limit states – tubular members, tubular joints and conical transitions

7.3.1 General

7.3.1.1 Tubular members shall be checked according to recognised standards. 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. The following standards are relevant for checking tubular member strength: Classification Notes 30.1 Sec.2 (Compact cross sections), Eurocode 3/EN 1993-1-1 and EN 1993-1-6, ISO19902 (D/t < 120) or NORSOK N-004 (D/t < 120). For interaction between local shell buckling and column buckling and for effect of external pressure, DNV-RP-C202 may be used.

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

\[
\frac{D}{t} \leq \sqrt{\frac{E}{f_y}}
\]

- \( E \) = modulus of elasticity
- \( f_y \) = minimum yield strength

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

7.3.1.2 Tubular members with external pressure, tubular joints and conical transitions may be checked according NORSOK N-004.

7.3.1.3 The material factor \( \gamma_M \) for tubular structures is 1.10.

7.3.1.4 For global buckling of towers and monopiles, the material factor \( \gamma_M \) shall be 1.2 as a minimum, see IEC61400-1.

7.3.1.5 The parametric formulas for shell buckling in EN 1993-1-6, based on membrane theory and applicable to tubular steel towers with D/t < 250, include a bias that may be accounted for by reducing the material factor \( \gamma_M \) for buckling to 1.1, when these formulas are used for assessment of global buckling.

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

7.4.1 General

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

7.4.1.2 Buckling checks may be performed according to Classification Notes 30.1.

7.4.1.3 Capacity checks may be performed according to recognised standards such as EN 1993-1-1 or AISC LRFD Manual of Steel Construction.
7.4.1.4 The material factors according to Table 7-2 shall be used if EN 1993-1-1 is used for calculation of structural resistance.

<table>
<thead>
<tr>
<th>Type of calculation</th>
<th>Material factor 1)</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistance of Class 1, 2 or 3 cross sections</td>
<td>( \gamma_M^0 )</td>
<td>1.10</td>
</tr>
<tr>
<td>Resistance of Class 4 cross sections</td>
<td>( \gamma_M^1 )</td>
<td>1.10</td>
</tr>
<tr>
<td>Resistance of members to buckling</td>
<td>( \gamma_M^1 )</td>
<td>1.10</td>
</tr>
</tbody>
</table>

1) Symbols according to EN 1993-1-1.

7.5 Ultimate limit states – special provisions for plating and stiffeners

7.5.1 Scope

7.5.1.1 The requirements in E will normally give minimum scantlings to plate and stiffened panels with respect to yield.

7.5.1.2 The buckling stability of plates may be checked according to DNV-RP-C201.

7.5.2 Minimum thickness

The thickness of plates should not be less than:

\[
t = \frac{14.3t_0}{\sqrt{f_{yd}}}(\text{mm})
\]

- \( f_{yd} \) = design yield strength \( f_y / \gamma_M \)
- \( f_y \) is the minimum yield stress (N/mm\(^2\)) as given in Sec.6 Table 6-3
- \( t_0 \) = 7 mm for primary structural elements
- = 5 mm for secondary structural elements
- \( \gamma_M \) = material factor for steel
- = 1.10.

7.5.3 Bending of plating

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

\[
t = \frac{15.8k_a k_r s \sqrt{p_d}}{\sqrt{\sigma_{pd1} k_{pp}}} (\text{mm})
\]

- \( k_a \) = correction factor for aspect ratio of plate field
- = (1.1 – 0.25 s/l)\(^2\)
- = maximum 1.0 for s/l = 0.4
- = minimum 0.72 for s/l = 1.0
- \( k_r \) = correction factor for curvature perpendicular to the stiffeners
- = (1 – 0.5 s/r_c)
- \( r_c \) = radius of curvature (m)
- \( s \) = stiffener spacing (m), measured along the plating
- \( p_d \) = design pressure (kN/m\(^2\)) as given in Sec.4
- \( \sigma_{pd1} \) = design bending stress
- = 1.3 (f_{yd} – \( \sigma_{jd} \)), but less than \( f_y / \gamma_M \)
- \( \sigma_{jd} \) = equivalent design stress for global in-plane membrane stress
- \( k_{pp} \) = fixation parameter for plate
The design bending stress $\sigma_{pd1}$ is given as a bi-linear capacity curve.

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

### 7.5.4 Stiffeners

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

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

- $l$ = stiffener span (m)
- $k_m$ = bending moment factor, see Table 7-4
- $\sigma_{pd2}$ = design bending stress
- $\alpha$ = angle between the stiffener web plane and the plane perpendicular to the plating.

#### 7.5.4.2 The formula given in [7.5.4.1] shall be regarded as the requirement about an axis parallel to the plating. As an approximation the requirement for standard section modulus for stiffeners at an oblique angle with the plating may be obtained if the formula in [7.5.4.1] is multiplied by the factor:

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

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

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

In such cases the section modulus of the stiffener calculated as indicated in [7.5.4.1] is normally to be based on the following parameter values:

- $k_m = 8$
- $k_{ps} = 0.9$

The stiffeners should normally be snipped with an angle of maximum $30^\circ$.

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

### 7.6 Ultimate limit states – special provisions for girders and girder systems

#### 7.6.1 Scope

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

#### 7.6.1.2 The buckling stability of girders may be checked according to DNV-RP-C201.
7.6.2 Minimum thickness

7.6.2.1 The thickness of web and flange plating is not to be less than given in [7.5.2] and [7.5.3].

7.6.3 Bending and shear

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

7.6.3.2 When boundary conditions for individual girders are not predictable due to dependence on adjacent structures, direct calculations according to the procedures given in [7.6.7] will be required.

7.6.3.3 The section modulus and web area of the girder shall be taken in accordance with particulars as given in [7.6.6] and [7.6.7]. Structural modelling in connection with direct stress analysis shall be based on the same particulars when applicable.

7.6.4 Effective flange

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

$$b_e = C_e b$$

- $C_e$ = as given in Figure 7-1 for various numbers of evenly spaced point loads ($N_p$) on the span
- $b$ = full breadth of plate flange e.g. span of the stiffeners supported by the girder with effective flange $b_e$, see also [7.6.6.2].
- $l_0$ = distance between points of zero bending moments (m)
  - $S$ for simply supported girders
  - $0.6 S$ for girders fixed at both ends

$S$ = girder span as if simply supported, see also [7.6.6.2].

![Figure 7-1](#)

Graphs for the effective flange parameter $C_e$

7.6.5 Effective web

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

7.6.6 Strength requirements for simple girders

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

- net section modulus according to [7.6.6.2]
- net web area according to [7.6.6.3].
7.6.6.2 Section modulus:

\[ Z_{g} = \frac{S^{2}bp_{d}}{k_{m}\sigma_{pd2}} \cdot 10^{6} \text{ (mm}^{3} \text{)} \]

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

b = breadth of load area (m) (plate flange) b may be determined as:

\[ b = 0.5 \left( l_{1} + l_{2} \right) \text{ (m)}, \]

where \( l_{1} \) and \( l_{2} \) are the spans of the supported stiffeners, or distance between girders.

\( k_{m} \) = bending moment factor \( k_{m} \)-values in accordance with Table 7-3 may be applied.

\( \sigma_{pd2} \) = design bending stress.

\( \sigma_{pd} \) = equivalent design stress for global in-plane membrane stress.

7.6.6.3 Net web area:

\[ A_{W} = \frac{k_{r}Sbp_{d} - N_{S}P_{pd}}{\tau_{p}} \cdot 10^{3} \text{ (mm}^{2} \text{)} \]

\( k_{r} \) = shear force factor \( k_{r} \)-may be in accordance with [7.6.6.4].

\( N_{S} \) = number of stiffeners between considered section and nearest support. The \( N_{S} \)-value is in no case to be taken greater than \((N_{p}+1)/4\).

\( N_{p} \) = number of supported stiffeners on the girder span.

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

\( \tau_{p} \) = \( 0.5 f_{yd} \) (N/mm²).

7.6.6.4 The \( k_{m} \) and \( k_{r} \)-values referred to in [7.6.6.2] and [7.6.6.3] may be calculated according to general beam theory. In Table 7-3, \( k_{m} \) and \( k_{r} \)-values are given for some defined load and boundary conditions. Note that the smallest \( k_{m} \)-value shall be applied to simple girders. For girders where brackets are fitted or the flange area has been partly increased due to large bending moment, a larger \( k_{m} \)-value may be used outside the strengthened region.
7.6.7 Complex girder system

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

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

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

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

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

7.6.7.5 The most unfavourable of the loading conditions given in Sec.4 shall be applied.

7.6.7.6 For girders taking part in the overall strength of the unit, stresses due to the design pressures given in Sec.4 shall be combined with relevant overall stresses.

7.7 Ultimate limit states – slip-resistant bolt connections

7.7.1 General

7.7.1.1 The requirements in G give the slip capacity of pre-tensioned bolt connections with high-strength bolts.
7.7.1.2 A high-strength bolt is defined as a bolt that has an ultimate tensile strength larger than 800 N/mm² and a yield strength which as a minimum is 80% of the ultimate tensile strength.

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

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

$$R_d \geq F_d$$

7.7.1.5 In addition, the slip resistant connection shall have the capacity to withstand ULS and ALS loads as a bearing bolt connection. The capacity of a bolted connection may be determined according to international recognised standards which give equivalent level of safety such as EN 1993-1-1 or AISC LRFD Manual of Steel Construction.

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

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

$k_s$ = hole clearance factor  
= 1.00 for standard clearances in the direction of the force  
= 0.85 for oversized holes  
= 0.70 for long slotted holes in the direction of the force

$n$ = number of friction interfaces  
$\mu$ = friction coefficient  
$\gamma_{Ms}$ = 1.25 for standard clearances in the direction of the force  
= 1.4 for oversize holes or long slotted holes in the direction of the force  
= 1.1 for design shear forces with load factor 1.0.

$F_{pd}$ = design preloading force.

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

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

$f_{ub}$ = ultimate tensile strength of the bolt  
$A_s$ = tensile stress area of the bolt (net area in the threaded part of the bolt).

7.7.1.8 The design value of the friction coefficient $\mu$ is dependent on the specified class of surface treatment as given in DNV-OS-C401 Sec.7. The value of $\mu$ shall be taken according to Table 7-4.

<table>
<thead>
<tr>
<th>Surface category</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.5</td>
</tr>
<tr>
<td>B</td>
<td>0.4</td>
</tr>
<tr>
<td>C</td>
<td>0.3</td>
</tr>
<tr>
<td>D</td>
<td>0.2</td>
</tr>
</tbody>
</table>

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

7.7.1.10 Provided the contact surfaces have been treated in conformity with DNV-OS-C401 Sec.7, the surface treatments given in Table 7-5 may be categorised without further testing.
7.7.1.11 Normal clearance for fitted bolts shall be assumed if not otherwise specified. The clearances are defined in Table 7-6.

<table>
<thead>
<tr>
<th>Clearance type</th>
<th>Clearance</th>
<th>Bolt diameter d (maximum)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard</td>
<td>1</td>
<td>12 and 14</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>16 to 24</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>27 to 36</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>42 to 48</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>64</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>84</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>104</td>
</tr>
<tr>
<td>Oversized</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>14 to 22</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>27</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>54</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>64</td>
</tr>
</tbody>
</table>

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

7.7.1.13 The nominal sizes of short slotted holes for slip resistant connections shall not be greater than given in Table 7-7.

<table>
<thead>
<tr>
<th>Maximum size mm</th>
<th>Bolt diameter d (maximum) mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d + 1) by (d + 4)</td>
<td>12 and 14</td>
</tr>
<tr>
<td>(d + 2) by (d + 6)</td>
<td>16 to 22</td>
</tr>
<tr>
<td>(d + 2) by (d + 8)</td>
<td>24</td>
</tr>
<tr>
<td>(d + 3) by (d + 10)</td>
<td>27 and larger</td>
</tr>
</tbody>
</table>

7.7.1.14 The nominal sizes of long slotted holes for slip resistant connections shall not be greater than given in Table 7-8.

<table>
<thead>
<tr>
<th>Maximum size mm</th>
<th>Bolt diameter d (maximum) mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d + 1) by 2.5 d</td>
<td>12 and 14</td>
</tr>
<tr>
<td>(d + 2) by 2.5 d</td>
<td>16 to 24</td>
</tr>
<tr>
<td>(d + 3) by 2.5 d</td>
<td>27 and larger</td>
</tr>
</tbody>
</table>

7.7.1.15 Long slots in an outer ply shall be covered by cover plates of appropriate dimensions and thickness. The holes in the cover plate shall not be larger than standard holes.

7.8 Ultimate limit states – welded connections

7.8.1 General

The requirements in this subsection apply to types and sizes of welds.
7.8.2 Types of welded steel joints

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

7.8.2.2 The connection of a plate abutting on another plate in a tee joint or a cross joint may be made as indicated in Figure 7-2.

7.8.2.3 The throat thickness of the weld is always to be measured as the normal to the weld surface, as indicated in Figure 7-2 d.

7.8.2.4 The type of connection should be adopted as follows:

a) Full penetration weld
   Important cross connections in structures exposed to high stress, especially dynamic, e.g. for special areas and fatigue utilised primary structure. All external welds in way of opening to open sea e.g. pipes, sea-chests or tee-joints as applicable.

b) Partial penetration weld
   Connections where the static stress level is high. Acceptable also for dynamically stressed connections, provided the equivalent stress is acceptable, see [7.8.3.12].

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

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

— oil-tight and watertight connections
— connections at supports and ends of girders, stiffeners and pillars
— connections in foundations and supporting structures for machinery.
7.8.2.6 Intermittent fillet welds may be used in the connection of girder and stiffener webs to plate and girder flange plate, respectively, where the connection is moderately stressed. With reference to Figure 7-3, the various types of intermittent welds are as follows:

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

7.8.2.7 Where intermittent welds are accepted, scallop welds shall be used in tanks for water ballast or fresh water. Chain and staggered welds may be used in dry spaces and tanks arranged for fuel oil only.
7.8.2.8 Slot welds, see Figure 7-4, may be used for connection of plating to internal webs, where access for welding is not practicable. The length of slots and distance between slots shall be considered in view of the required size of welding.

7.8.2.9 Lap joints as indicated in Figure 7-5 may be used in end connections of stiffeners. Lap joints should be avoided in connections with dynamic stresses.

7.8.3 Weld size

7.8.3.1 The material factors $\gamma_{Mw}$ for welded connections are given in Table 7-9.

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Material factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS</td>
<td>1.25</td>
</tr>
</tbody>
</table>

7.8.3.2 If the yield stress of the weld deposit is higher than that of the base metal, the size of ordinary fillet weld connections may be reduced as indicated in [7.8.3.4].

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

7.8.3.3 Welding consumables used for welding of normal steel and some high strength steels are assumed to give weld deposits with characteristic yield stress $\sigma_{fy}$ as indicated in Table 7-10. If welding consumables with deposits of lower yield stress than specified in Table 7-10 are used, the applied yield strength shall be clearly informed on drawings and in design reports.
7.8.3.4 The size of some weld connections may be reduced:
— corresponding to the strength of the weld metal, $f_w$:

$$ f_w = \left( \frac{\sigma_{fw}}{235} \right)^{0.75} $$

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

$$ f_r = \left( \frac{f_y}{\sigma_{fw}} \right)^{0.75} $$

minimum 0.75

$f_y$ = characteristic yield stress of base material, abutting plate (N/mm$^2$)

$\sigma_{fw}$ = characteristic yield stress of weld deposit (N/mm$^2$)

Ordinary values for $f_w$ and $f_r$ for normal strength and high-strength steels are given in Table 7-10. When deep penetrating welding processes are applied, the required throat thicknesses may be reduced by 15% provided that sufficient weld penetration is demonstrated.

7.8.3.5 Conversions between NV grades as used in Table 7-10 and steel grades used in the EN 10025-2 standard are given in Sec.6.

7.8.3.6 Where the connection of girder and stiffener webs and plate panel or girder flange plate, respectively, are mainly shear stressed, fillet welds as specified in [7.8.3.7] to [7.8.3.9] should be adopted.

<table>
<thead>
<tr>
<th>Table 7-10 Strength ratios, $f_w$ and $f_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Base metal</strong></td>
</tr>
<tr>
<td><strong>Designation</strong></td>
</tr>
<tr>
<td><strong>Strength group</strong></td>
</tr>
<tr>
<td>Normal strength steels</td>
</tr>
<tr>
<td>High strength steels</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

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

$$ t_w = \frac{0.43 \times f_r \times t_0}{t_w} (\text{mm}), \text{minimum } 3 \text{ mm} $$

$f_r$ = strength ratio as defined in [7.8.3.4]

$t_0$ = net thickness (mm) of abutting plate.

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

7.8.3.8 The throat thickness of intermittent welds may be as required in [7.8.3.7] for double continuous welds provided the welded length is not less than:

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

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

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

— for chain welds and scallop welds:
$t_w = 0.6 f_r t_0 \text{ (mm)}$

$t_0 = \text{ net thickness abutting plate:}

\begin{align*}
\text{— for staggered welds:} \\
&t_w = 0.75 f_r t_0 \text{ (mm)}
\end{align*}

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

7.8.3.10 In structural parts where dynamic stresses or high static tensile stresses act through an intermediate plate, see Figure 7-2, penetration welds or increased fillet welds shall be used.

7.8.3.11 When the abutting plate carries dynamic stresses, the connection shall fulfil the requirements with respect to fatigue, see [7.10].

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

$$t_w = \frac{1.36}{f_w} \left[ 0.2 + \left( \frac{\sigma_d}{320} - 0.25 \right) \frac{r}{t_0} \right] \text{ (mm)}$$

minimum 3 mm.

$f_w = \text{ strength ratio as defined in [7.8.3.4]}$

$\sigma_d = \text{ calculated maximum design tensile stress in abutting plate (N/mm}^2\text{)}$

$r = \text{ root face (mm), see Figure 7-2 b}$

$t_0 = \text{ net thickness (mm) of abutting plate.}$

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

\begin{itemize}
  \item welded directly to the web plate on one or both sides of the stiffener
  \item connected by single- or double-sided lugs
  \item with stiffener or bracket welded on top of frame
  \item a combination of the ways listed above.
\end{itemize}

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

7.8.3.14 Various standard types of connections between stiffeners and girders are shown in Figure 7-6.
7.8.3.15 Connection lugs should have a thickness not less than 75% of the web plate thickness.

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

\[ a_0 = \sqrt{5} \frac{c}{f_{yd}} \times 10^{-3} (l - 0.5s)sp_d \] (mm²)

- \( c \) = detail shape factor as given in Table 7-11
- \( f_{yd} \) = minimum yield design stress (N/mm²)
- \( l \) = span of stiffener (m)
- \( s \) = distance between stiffeners (m)
- \( p_d \) = design pressure (kN/m²).

<table>
<thead>
<tr>
<th>Table 7-11 Detail shape factor c</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type of connection</strong> (see Figure 7-6)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>a</td>
</tr>
<tr>
<td>b</td>
</tr>
<tr>
<td>c</td>
</tr>
</tbody>
</table>
The total weld area \( a \) is not to be less than:

\[
a = f_r a_0 \text{ (mm}^2\text{)}
\]

\( f_r \) = strength ratio as defined in [7.8.3.4]

\( a_0 \) = connection area (mm\(^2\)) as given in [7.8.3.16].

The throat thickness is not to exceed the maximum for scallop welds given in [7.8.3.9].

7.8.3.17 The weld connection between stiffener end and bracket is in principle to be designed such that the design shear stresses of the connection correspond to the design resistance.

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

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

7.8.3.20 The weld connection area of bracket to adjoining girders or other structural parts shall be based on the calculated normal and shear stresses. Double continuous welding shall be used. Where large tensile stresses are expected, design according to [7.8.3.10], [7.8.3.11], and [7.8.3.12] shall be applied.

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

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

\[
t_w = \frac{\tau_d}{260 f_w} t_0 \text{ (mm)}
\]

\( \tau_d \) = design shear stress in web plate (N/mm\(^2\))

\( f_w \) = strength ratio for weld as defined in [7.8.3.4]

\( t_0 \) = net thickness (mm) of web plate.

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

7.8.3.23 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.

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

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

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

**Guidance note:**

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

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

7.8.3.27 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.

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

\[
\sqrt{\sigma_{ld}^2 + 3 \left( f_{ld}^2 + \tau_{ld}^2 \right)} \leq \frac{f_u}{\gamma_{Mw}}
\]

and

\[
\sigma_{ld} \leq \frac{f_u}{\gamma_{Mw}}
\]

\( \sigma_{ld} \) = normal design stress perpendicular to the throat (including load factors)
7.9  Serviceability Limit States

7.9.1 General

Serviceability limit states for offshore steel structures are associated with:

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

7.9.2 Deflection criteria

7.9.2.1 For calculations in the serviceability limit states

\[ \gamma_M = 1.0. \]

7.9.2.2 Limiting values for vertical deflections should be given in the design brief. In lieu of such deflection criteria limiting values given in Table 7-13 may be used.
7.9.2.3 The maximum vertical deflection is:

\[ \delta_{\text{max}} = \delta_1 + \delta_2 - \delta_0 \]

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

Figure 7-8
Definitions of vertical deflections

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

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

7.10 Fatigue limit states

7.10.1 Fatigue limit state

7.10.1.1 In this subsection, requirements are given for design against fatigue failure. Reference is made to DNV-RP-C203 for practical details with respect to fatigue design of offshore structures.

7.10.1.2 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. Prediction of fatigue lives is used in fatigue design to fulfill this aim. Prediction 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.

7.10.1.3 The resistance against fatigue 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 S. The S-N curve is usually based on fatigue tests in the laboratory. For interpretation of S-N curves from fatigue tests, the fatigue failure is defined to have occurred when a fatigue crack has grown through the thickness of the structure or structural component.
7.10.1.4 The characteristic S-N curve shall in general be taken as the curve that corresponds to the 2.3% quantile of N for given S, i.e. the S-N curve that provides 97.7% probability of survival.

7.10.1.5 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.

7.10.1.6 To ensure that the structure will fulfill the intended function, a fatigue assessment shall be carried out for each individual member, which is subjected to fatigue loading. Where appropriate, the fatigue assessment shall be supported by a detailed fatigue analysis.

**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.

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

7.10.1.7 Corrosion allowance shall be taken into account by decreasing the nominal wall thickness in fatigue limit state analyses.

**Guidance note:**
Fatigue calculations can be based on a steel wall thickness equal to the nominal thickness reduced by half the corrosion allowance over the full service life, in which 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 and (2) 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 in [11.2.2].

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.

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

7.10.2 Characteristic S-N curves

7.10.2.1 The fatigue strength of welded joints is to some extent dependent on plate thickness. This effect is 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 curve can be taken as

\[
\log_{10} N = \log_{10} a - m \log_{10} \left( \frac{\Delta \sigma}{t_{\text{ref}}} \right)^k
\]

in which

- \(N\) = fatigue life, i.e. number of stress cycles to failure at stress range \(\Delta \sigma\)
- \(\Delta \sigma\) = stress range in units of MPa
- \(m\) = negative slope of S-N curve on logN-logS plot
- \(\log a\) = intercept of logN axis
- \(t_{\text{ref}}\) = reference thickness, \(t_{\text{ref}} = 32\) mm for tubular joints,
  \(t_{\text{ref}} = 25\) mm for welded connections other than tubular joints, such as girth welds
- \(t\) = thickness through which the potential fatigue crack will grow; \(t = t_{\text{ref}}\) shall be used in expression when \(t < t_{\text{ref}}\)
- \(k\) = thickness exponent, also known as scale exponent, see Table 7-14.

The S-N curves for the most frequently used structural details in steel support structures for offshore wind turbines are given in Table 7-14. The use of the S-N curves in Table 7-14 is an option. They can be used when project-specific or manufacturer-specific data are not available for all ranges of applicability of S-N curves. Some of the S-N curves in Table 7-14 depend on the attachment length \(l\), see Figure 7-9 for definition.
**Figure 7-9**
Definition of attachment length for welded attachments referenced in Table 7-14

<table>
<thead>
<tr>
<th>Structural detail</th>
<th>Curve</th>
<th>Environment</th>
<th>In air</th>
<th>In seawater with cathodic protection</th>
<th>Free corrosion</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td><strong>logₐₐ</strong></td>
<td><strong>m</strong></td>
<td><strong>Range of validity</strong></td>
<td><strong>k</strong></td>
</tr>
<tr>
<td>Weld in tubular joint</td>
<td>T</td>
<td>12.164 3</td>
<td>N &lt; 10⁷</td>
<td>0.25</td>
<td>11.764 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.606 5</td>
<td>N &gt; 10⁷</td>
<td>0.25</td>
<td>15.606 5</td>
</tr>
<tr>
<td>Butt weld and tubular girth weld, weld toe (1)</td>
<td>D</td>
<td>12.164 3</td>
<td>N &lt; 10⁷</td>
<td>0.20</td>
<td>11.764 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.606 5</td>
<td>N &gt; 10⁷</td>
<td>0.20</td>
<td>15.606 5</td>
</tr>
<tr>
<td>Butt weld and tubular girth weld, weld root (1) (2)</td>
<td>F3</td>
<td>11.546 3</td>
<td>N &lt; 10⁷</td>
<td>0.25</td>
<td>11.146 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>14.576 5</td>
<td>N &gt; 10⁷</td>
<td>0.25</td>
<td>14.576 5</td>
</tr>
<tr>
<td>Non-load carrying welded attachments of length / in main stress direction (3)</td>
<td>E</td>
<td>12.010 3</td>
<td>N &lt; 10⁷</td>
<td>0.20</td>
<td>11.610 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.350 5</td>
<td>N &gt; 10⁷</td>
<td>0.20</td>
<td>15.350 5</td>
</tr>
<tr>
<td>Butt weld and tubular girth weld, weld root (1) (2)</td>
<td>F</td>
<td>11.855 3</td>
<td>N &lt; 10⁷</td>
<td>0.25</td>
<td>11.455 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.091 5</td>
<td>N &gt; 10⁷</td>
<td>0.25</td>
<td>15.091 5</td>
</tr>
<tr>
<td>F1</td>
<td></td>
<td>11.699 3</td>
<td>N &lt; 10⁷</td>
<td>0.25</td>
<td>11.299 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>14.832 5</td>
<td>N &gt; 10⁷</td>
<td>0.25</td>
<td>14.832 5</td>
</tr>
<tr>
<td>F3</td>
<td></td>
<td>11.546 3</td>
<td>N &lt; 10⁷</td>
<td>0.25</td>
<td>11.146 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>14.576 5</td>
<td>N &gt; 10⁷</td>
<td>0.25</td>
<td>14.576 5</td>
</tr>
</tbody>
</table>

1) For girth welds welded from both sides, the S-N curves for the weld toe apply at both sides. For girth welds welded from one side only, the S-N curves for the weld toe position apply to the side from which the weld has been welded up, and the S-N curves for the weld root apply to the other side.

2) Transverse butt weld on a temporary or permanent backing strip without fillet welds.

3) Examples of the attachment length / for different types of welds are given in Figure 7-9.

**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, can be taken from DNV-RP-C203 “Fatigue Strength Analyses of Offshore Steel Structures”.

---

**Figure 7-9**
Definition of attachment length for welded attachments referenced in Table 7-14
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 in Table 7-14 is that a high fabrication quality of the details is present, i.e. welding and NDT shall be in accordance with Inspection Category I and Structural Category ‘Special’ according to DNV-OS-C401 Ch.2 Sec.3 Tables C3, C4 and C5. For structural details in the tower, the requirement of NDT inspections in accordance with Inspection Category I is waived, see [6.1.3.9].

For S-N curves for plated structures, I-girders and other structural details than those covered by Table 7-14, reference is made to DNV-RP-C203.

The “free corrosion” S-N curves can 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---

7.10.2.2 The thickness-dependent modification of the stress range in [7.10.2.1] is intended to reflect that the actual size and geometry of the structural component considered are different from what the S-N data (logσ and m) in Table 7-14 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 and of the weld width in butt welds. Let \( L \) denote the weld attachment length and the weld width in these two types of joints as defined in Figure 7-10. For cruciform joints and for butt welds, the thickness \( t \) in the expression for the characteristic S-N curve can be replaced by an effective thickness \( t_{eff} \)

\[
    t_{eff} = \min\{4mm + 0.66 \cdot L, T\}
\]

where the parameters \( L \) and \( T \) are defined in Figure 7-10 and measured in units of mm. Note that \( L \) in this context has a slightly different definition than the attachment length \( l \) referenced in Table 7-14.

The effective thickness for a pile weld bead or a pile sleeve weld bead is identified as a special case for evaluation. The effective thickness for this case can be derived from the expression for \( t_{eff} \) by substituting the bead width \( w \) for \( L \).

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

7.10.2.3 Requirements for S-N curves are given in Table 7-15.

7.10.2.4 As a supplement to an S-N curve based fatigue calculation, a calculation of the fatigue life may be based on a fracture mechanics analysis, see DNV-RP-C203. App.E provides a method for calculation of the fatigue life for tubular connections (tubular joints and tubular girth welds) based on fracture mechanics. An alternative method for fracture mechanics calculations can be found in BS 7910.

7.10.3 Characteristic stress range distribution

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

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

The choice of wave theory to be applied for calculation of wave kinematics is to be made according to Sec.3. The wave theory depends much on the water depth. For water depths less than approximately 15 m, higher order stream function theory is to be applied. For water depths in excess of approximately 30 m, Stokes 5th order theory is to be applied.
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.

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.

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

7.10.3.3 Whenever appropriate, all stress ranges of the long-term stress range distribution shall be multiplied by a stress concentration factor (SCF). The SCF depends on the structural geometry. SCFs can be calculated from parametric equations or by finite element analysis. Guidance for finite element analysis is given in App.K.

If a characteristic value of misalignment is derived from a well-defined fabrication process, defined as an upper 5 percentile value, this value can be used for fatigue assessment together with equations for stress concentration factors from DNV-RP-C203 by putting $\delta_0 = 0.05t$.

**Guidance note:**
In wind farms, where the same joint or structural detail is repeated many times in many identical support structures, requirements to cost-effectiveness makes it particularly important to assess the SCFs accurately, and 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 DNV-RP-C203.

When finite element methods based on conventional rigid-joint frame models of beam elements are used to calculate SCFs for tubular joints, it is important to include local joint flexibilities. Such local joint flexibilities exist, but are not reflected in the rigid beam element connections of such frame models. For inclusion of local joint flexibilities, Buitrago’s parametric formulae shall be used. Details are given in App.B.

For multi-planar tubular joints for which the multi-planar effects are not negligible, the SCFs may either be determined by a detailed FEM analysis of each joint or by selecting the largest SCF for each brace among the values resulting from considering the joint to be a Y, X and K joint.

When conical stubs are used, the SCF may be determined by using 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 shall be applied.

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

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

5% of stress is assumed accounted for in the specified S-N curves.

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

7.10.3.4 For fatigue analysis of regions in base material not significantly affected by residual stresses due to welding, 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. An example of application is cut-outs in the base material. The mean stress $\sigma_m$ is the static stress including stress concentration factors. Let $\Delta\sigma$ denote the stress range including stress concentration factors. Prior to execution of the fatigue analysis, in which the long-term stress range distribution is applied together with the S-N curve for prediction of fatigue damage, the stress ranges may be multiplied by a reduction factor $f_m$ which is in general obtained from Figure 7-11:

![Figure 7-11](image-url)

**Figure 7-11**
Stress range reduction factor $f_m$ to be used with S-N curve for base material
The reduction factor $f_m$ is expressed as

$$f_m = \frac{\sigma_t + 0.6 \cdot |\sigma_c|}{\sigma_t + |\sigma_c|} \quad \text{for } -\Delta \sigma/2 < \sigma_m \leq \Delta \sigma/2$$

where

- $\sigma_t$ = maximum tensile stress, where tension is defined as positive
- $\sigma_c$ = maximum compressive stress, where compression is defined as negative
- $\sigma_m = (\sigma_t + \sigma_c)/2$ = mean stress
- $\Delta \sigma = \sigma_t - \sigma_c$ = stress range.

This implies in particular that $f_m$ is 1.0 when the material is in tension during the entire stress cycle, 0.8 when it is subject to zero-mean stress, and 0.0 when the material is in compression during the entire stress cycle.

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

7.10.3.5 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 mean stress $\sigma_m$ is the static notch stress including stress concentration factors. Let $\Delta \sigma$ denote the stress range including stress concentration factors. Prior to execution of the fatigue analysis, in which the long-term stress range distribution is applied together with the S-N curve for prediction of fatigue damage, the stress ranges may be multiplied by a reduction factor $f_m$ which can be obtained from Figure 7-12:

![Figure 7-12](image)

**Figure 7-12**

**Stress range reduction factor $f_m$ to be used with S-N curve for weld material**

This implies in particular that $f_m$ is 1.0 when the material is in tension during the entire stress cycle, 0.8 when it is in compression during the entire stress cycle, and 0.9 when it is subject to zero-mean stress.

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

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

**Guidance note:**

When the natural period of the wind turbine, support structure and foundation is less than or equal to 2.5 sec, a dynamic amplification factor $DAF$ may be applied to the wave load on the structure, when the wind turbine, support structure and foundation are modelled as a single-degree-of-freedom system

$$DAF = \frac{1}{\sqrt{(1-\Omega^2)^2 + (2\xi \Omega)^2}}$$

in which

- $\xi$ = damping ratio relative to critical damping
- $\Omega$ = ratio between applied frequency and natural frequency

When the natural period of the wind turbine, support structure and foundation is greater than 2.5 sec, a time domain analysis shall be carried out to determine the dynamic amplification factor.

Caution must be exercised when assessing the damping of jacket type support structures because of the limited experience with this type of structure being used for support of wind turbines. For jacket type structures, the damping ratio can often be chosen as 1% relative to critical damping. However, when a turbine and a tower are assembled on a stiff jacket structure, the first mode damping will be lower than 1% of critical damping. The vibration modes relevant for determination of dynamic amplification factors are typically the global sway modes, which can be excited by wave loading.

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---
7.10.3.7 The stress ranges in the stress range distribution must be compatible with the stress ranges of the S-N curve that the distribution is to be used with for fatigue damage predictions. At welds, where stress singularities are present and extrapolation needs to be applied to solve for the stress ranges, this implies that the same extrapolation procedure must 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 in this area due to the presence of the weld. In addition comes that 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. The hot spot stress at the weld root and the weld toe is established by extrapolation from stresses measured outside the notch zone. During testing for interpretation of the 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 must be established as hot spot stresses for the weld in question, i.e. the stresses in the welds must be established by extrapolation from stresses in the extrapolation points which are prescribed for the actual structural detail. Thus when a finite element analysis is used to establish the stresses in the welds from the applied loading, the stresses in the welds are to be found by extrapolation from the stresses that are calculated by the analysis in the prescribed extrapolation points.

Reference is made to DNV-RP-C203 for more details.

---end---of---Guidance---note---

7.10.4 Characteristic cumulative damage and design cumulative damage

7.10.4.1 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

$$D_C = \sum_{i=1}^{I} \frac{n_{C,i}}{N_{C,i}}$$

in which

- $D_C$ = characteristic cumulative damage
- $I$ = total number of stress range blocks in a sufficiently fine, chosen discretisation of the stress range axis
- $n_{C,i}$ = number of stress cycles in the $i$th stress block, interpreted from the characteristic long-term distribution of stress ranges
- $N_{C,i}$ = number of cycles to failure at stress range of the $i$th stress block, interpreted from the characteristic S-N curve

---end---of---Guidance---note---

7.10.4.2 The design cumulative damage $D_D$ is obtained by multiplying the characteristic cumulative damage $D_C$ by the design fatigue factor DFF

$$D_D = DFF \cdot D_C$$

7.10.5 Design fatigue factors

7.10.5.1 The design fatigue factor (DFF) is a partial 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 can be applied as a divisor on the calculated characteristic fatigue life to obtain the calculated design fatigue life.

---end---of---Guidance---note---

7.10.5.2 The design fatigue factors in Table 7-15 are valid for 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 damaged condition with failure in the actual joint as the specified damage. The design fatigue factors in Table 7-15 depend on the location of the structural detail, of the accessibility for inspection and repair, and of the type of corrosion protection. The relation between the level of inspection and
the requirement for DFF is detailed in [13.3.5.2]. 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 DFF = 2.0 in the atmospheric zone reflects an assumption of no or irregular inspections, regardless of accessibility. DFF = 1.0 may be used in the atmospheric zone if an inspection plan is established with documented inspection method and inspection intervals that will result in the same safety level as DFF = 2.0 without inspections.

Guidance note:

Wind turbines are normally unmanned, such that the consequences of failure can be reckoned as low. For towers which are not affected by low-cycle fatigue and which are to be designed as inspection-free with DFF=2.0, the design fatigue damage may alternatively be calculated as

\[ D_D = \sum_{i=1}^{I} \frac{n_{C,i}}{N_{D,i}} \]

in which

- \( D_D \) = design cumulative fatigue damage
- \( I \) = total number of stress range blocks in a sufficiently fine, chosen discretisation of the stress range axis
- \( n_{C,i} \) = number of stress cycles in the \( i \)th stress block, interpreted from the characteristic long-term distribution of stress ranges
- \( N_{D,i} \) = number of cycles to failure at the design stress range \( \Delta \sigma_{d,i} = \gamma_m \Delta \sigma_i \) of the \( i \)th stress block, interpreted from the characteristic S-N curve
- \( \gamma_m = 1.15 \)
- \( \Delta \sigma_i \) = stress range of the \( i \)th stress block in the characteristic long-term distribution of stress ranges.

For further details regarding fatigue design of towers, reference is made to IEC 61400-1 and IEC 61400-3.

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---
7.10.5.3 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. For further details, reference is made to DNV-RP-C203, Appendix D.

7.10.6 Design requirement

The design criterion is

\[ D_D \leq 1.0 \]

7.10.7 Improved fatigue performance of welded structures by grinding

7.10.7.1 The fatigue performance of welds in tubular joints 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. If the grinding is performed in accordance with Figure 7-13, 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 reduced from 0.25 to 0.20. References is made to DNV-RP-C203, Appendix D.

Figure 7-13
Weld toe grinding

7.10.7.2 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 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 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.

If weld toe grinding is performed on “old” joints according to the above specification, these joints can be considered as ‘newborn’ when their fatigue lives are to be predicted.

7.10.7.3 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 7-14.
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.

If the grinding is performed as shown to the right in Figure 7-14 and 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 DNV-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 depth of grinding 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.

**Guidance note:**
The following improved S-N curves can be applied for girth welds if grinding is carried out according to the above specifications:

For grinded girth welds in air:

\[
\log a = 12.592 \text{ and } m = 3 \text{ for } N < 10^7, k = 0.05 \\
\log a = 16.320 \text{ and } m = 5 \text{ for } N > 10^7, k = 0.05
\]

(Curve ‘C’)

For girth welds in seawater with adequate cathodic protection:

\[
\log a = 12.192 \text{ and } m = 3 \text{ for } N < 10^6, k = 0.05 \\
\log a = 16.320 \text{ and } m = 5 \text{ for } N > 10^6, k = 0.05
\]

(Curve ‘C’)

Reference is made to DNV-RP-C203, Appendix D.
8.1 Introduction

8.1.1 General

8.1.1.1 For detailed design of offshore wind turbine concrete structures, DNV-OS-C502, “Offshore Concrete Structures” shall apply together with the provisions of this section. Alternatively, other standards can be used as specified in [1.1.4]. It is the responsibility of the designer to document that the requirements in [1.1.4] are met.

8.1.1.2 The loads that govern the design of an offshore wind turbine concrete structure are specified in Sec.4 and Sec.5. SLS loads for offshore wind turbine concrete structures are defined in this section. Details regarding the process of determining the load effects within the concrete structure can be found in DNV-OS-C502.

8.1.1.3 Sec.8 in general provides requirements and guidance which are supplementary to the provisions of DNV-OS-C502. Hence, Sec.8 shall be considered application text for DNV-OS-C502 with respect to offshore wind turbine concrete structures. For all design purposes, the user should always refer to the complete description in DNV-OS-C502 together with this section.

8.1.1.4 Sec.8 in particular provides requirements and guidance for how to use EN standards as a supplement to DNV standards for design of offshore concrete structures. Such use of EN standards as a supplement to DNV standards shall be carried out according to the requirements in [1.1.4].

8.1.2 Material

The requirements to materials given in DNV-OS-C502 Sec.4 and in Sec.6 of this standard shall apply for structures designed in accordance with this section.

8.1.3 Composite structures

For design of composite structures such as pile-to-sleeve connections and similar connections, the requirements given in DNV-OS-C502 Sec.6 A500 shall be supplemented by the requirements given in Sec.9 of this standard.

8.2 Design principles

8.2.1 Design material strength

8.2.1.1 In design by calculation according to DNV-OS-C502 together with this standard, the design material strength shall be taken as a normalized value of the in-situ strength divided by a material factor $\gamma_m$ (ref. DNV-OS-C502 Sec.4 C300 and [8.2.1.3] in this standard).

Guidance note:
It is important to note that the partial safety factor $\gamma_m$ 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. The normalized compressive strength and the characteristic compressive strength are not necessarily the same.

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

8.2.1.2 For wind turbine structures, Young’s Modulus for concrete shall be taken equal to the normalized value $E_{cn}$, both for the serviceability limit state and for the fatigue limit state (ref. DNV-OS-C502 Sec.6 C111).

8.2.1.3 The material factors, $\gamma_m$, for concrete and reinforcement for offshore wind turbine concrete structures are given in Table 8-1.

Guidance note:
It is noted that the requirements to the material factor for ULS design as specified in Table 8-1 are somewhat lower than the corresponding requirements in DNV-OS-C502. This difference merely reflects that DNV-OS-C502 is meant for design to high safety class (manned structures with large consequence of failure) whereas DNV-OS-J101 aims at design to normal safety class (unmanned structures, structures with small consequences of failure).

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---
8.3 Basis for design by calculation

8.3.1 Concrete grades and in-situ strength of concrete

In DNV-OS-C502 Sec.4 C300 normal weight concrete has grades identified by the symbol C and lightweight aggregate concrete grades are identified by the symbol LC. The grades are defined in DNV-OS-C502 Sec.4 Table 9-1 as a function of the Characteristic Compressive Cylinder strength of the concrete, $f_{ck}$.

8.4 Bending moment and axial force (ULS)

8.4.1 General

For design according to EN 1992-1-1:2004 in the Ultimate Limit State the strength definition can be used from the EN standard together with the general material factors (ref. EN 1992-1-1: 2004, Table 2.1N).

8.5 Fatigue limit state

8.5.1 General

Fatigue design shall be carried out in accordance with DNV-OS-C502 Sec.6M. If structures are not planned to be inspected the cumulative fatigue damage shall be limited to 0.33.

8.6 Accidental limit state

8.6.1 General

According to DNV-OS-C502, structures classified in Safety Classes 2 and 3 (see DNV-OS-C502 Sec.2 A300) shall be designed in such a way that an accidental load will not cause extensive failure. Support structures and foundations for offshore wind turbines are in this standard defined to belong to Safety Class 2.

8.7 Serviceability limit state

8.7.1 Durability

8.7.1.1 When the formula in DNV-OS-C502 Sec.6 O306 for the nominal crack width ($w_k = w_{ck} \cdot \left( \frac{c_1}{c_2} \right) > 0.7 \cdot w_{ck}$) is used, the value for $c_2$ shall be taken as given below:

$c_2 = \text{actual nominal concrete cover to the outermost reinforcement (e.g. stirrups)}$

8.7.1.2 For offshore wind turbine concrete structures, the load for crack width calculations is to be taken as the maximum characteristic load that can be defined among the wind and wave climate combinations used for the FLS load cases. The wind and wave climate combinations used for the FLS load cases are specified in Sec.4 Table 4-5. The characteristic load for a particular combination of wind climate and wave climate is defined as the 90% quantile in the distribution of the maximum load in a 10-minute reference period with this particular climate combination. Based on this, the following procedure can be used to determine the load for crack width calculations:

<table>
<thead>
<tr>
<th>Limit State</th>
<th>ULS</th>
<th>FLS</th>
<th>ALS</th>
<th>SLS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced concrete/grout</td>
<td>$\gamma_c$</td>
<td>1.45</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Steel reinforcement</td>
<td>$\gamma_s$</td>
<td>1.10</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Plain concrete/grout, fibre reinforced concrete/grout</td>
<td>$\gamma_c$</td>
<td>1.45</td>
<td>1.50</td>
<td>1.20</td>
</tr>
</tbody>
</table>

1) Design with these material factors allows for tolerances in accordance with DNV-OS-C502 Sec.6 C500 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 DNV-OS-C502 Sec.6 C500 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% is to be accounted for in the resistance calculations. Alternatively, the material factors may be taken according to those given under 3).

2) When the design is to 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 can be used. When these factors are used, then any geometric deviations from the “approved for construction” drawings must 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 DNV-OS-C502 Sec.6 T700).
1) For each considered applicable combination of wind climate and wave climate, at least 6 10-minutes time series of load (or load effect) in relevant cross sections shall be calculated by simulation with different seeds.

2) From each of the time series for a particular cross section and a particular combination of wind and wave climate, the maximum load or load effect shall be interpreted.

3) For each relevant cross section and particular combination of wind and wave climate, the mean value and the standard deviation of the interpreted six or more maxima (one from each simulated time series of load or load effect) shall be calculated.

4) For each relevant cross section and particular combination of wind and wave climate, the characteristic load can be calculated as mean value + 1.28 \times \text{standard deviation}.

5) For each relevant cross section considered, the load for crack width calculation shall be taken as the maximum characteristic load over all applicable combinations of wind and wave climate considered.

**Guidance note:**

Usually it will be sufficient to consider the production and idling load cases i.e. Load Cases 1.2 and 6.4 according to Sec.4, Table 4-5.

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

8.7.1.3 In order to avoid permanent deformations, the strain in the reinforcement shall be calculated for the characteristic extreme ULS load and it shall be substantiated that this strain does not exceed 90% of the yield strain of the reinforcement.

### 8.7.2 Crack width calculation

#### 8.7.2.1

Crack widths shall be calculated in accordance with the method described in DNV-OS-C502 Sec.6 O800 and DNV-OS-C502 App.E.

Let \( \varepsilon_{sm} \) denote the mean principal tensile strain in the reinforcement over the crack’s influence length at the outer layer of the reinforcement. Let \( \varepsilon_{cm} \) denote the mean stress-dependent tensile strain in the concrete at the same layer and over the same length as \( \varepsilon_{sm} \).

For estimation of \( (\varepsilon_{sm} - \varepsilon_{cm}) \) the following expression shall be used:

\[
(\varepsilon_{sm} - \varepsilon_{cm}) = \frac{\sigma_s}{E_{sk}} \cdot (1 - \beta_s \cdot \frac{\sigma_{sr}}{\sigma_s})
\]

where:

- \( E_{sk} \) = characteristic value of Young's modulus of steel reinforcement
- \( \sigma_s \) = the stress in reinforcement at the crack calculated for the actual load.
- \( \sigma_{sr} \) = 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 DNV-OS-C502 Sec.4 Table C1.
- \( \beta_s \leq \frac{\sigma_s}{\sigma_{sr}} \)
- \( \beta_s = 0.4 \).

#### 8.7.2.2

For guidance on how to calculate the free shrinkage strain of the concrete, \( \varepsilon_{cs} \), reference is made to NS 3473:2003, Section A9.3.2.

#### 8.7.2.3

For design according to EN standards the crack width formulae in EN 1992-1-1:2004 can be used with the following prescribed coefficient values which will yield results similar to results according to DNV-OS-C502:

a) \( h_{cef} \) shall be defined according to DNV-OS-C502 App.E
b) \( k_2 \) shall be defined as \( K_c \) according to DNV-OS-C502 App.E
c) \( k_3 \) shall as a minimum be taken as 1.36
d) \( k_4 \) shall be taken as 0.425

**Guidance note:**

For crack width calculation according to EN 1992-1-1:2004 with the prescribed coefficient values, the crack width criterion can be taken according to DNV-OS-C502.

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

### 8.7.3 Other serviceability limit states

Limitations of stresses in the concrete (ref. DNV-OS-C502 Sec.6 O903) are also governing for concrete wind turbine support structures with normal reinforcement. The SLS load to be considered is the load defined in [8.7.1.2] for the crack width calculation in [8.7.2.1].
8.8 Detailing of reinforcement

8.8.1 Positioning
All shear reinforcements and stirrups shall be anchored outside the main reinforcement (i.e. they shall encircle the reinforcement).

8.9 Corrosion control and electrical earthing

8.9.1 Corrosion control

8.9.1.1 Requirements for corrosion protection arrangement and equipments are generally given in Section 11. Special evaluations relevant for offshore concrete structures are given in 8.9.1.2 and in DNV-OS-C502 Sec.6 R100 to R400.

8.9.1.2 Concrete rebars and prestressing tendons are adequately protected by the concrete itself, i.e. provided adequate coverage and adequate type and quality of the aggregate. However, rebar portions freely exposed to seawater in case of concrete defects and embedment plates, penetration sleeves and various supports (e.g. appurtenances) which are freely exposed to seawater or to the marine atmosphere will normally 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 8.9.2.

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

8.9.2 Electrical earthing
All metallic components in an offshore support 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 reference is made to EN 62305.

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

8.10 Construction

8.10.1 General

8.10.1.1 Construction shall be performed according to DNV-OS-C502 Sec.7, if necessary together with other relevant standards as stated in DNV-OS-C502 Sec.7 A201.

8.10.1.2 For structures designed according to other standards systems (e.g. EN standards) the construction standards in the actual system shall be also be applied.

8.10.2 Inspection classes

8.10.2.1 In general, inspection class IC2, “Normal inspection”, (see DNV-OS-C502 Sec.7 D201) applies for offshore wind turbine concrete structures.

8.10.2.2 For construction according to EN 13670-1:2009, Inspection Class 2 applies for offshore wind turbine concrete structures.
SECTION 9 DESIGN AND CONSTRUCTION OF GROUTED CONNECTIONS

9.1 Introduction

9.1.1 General

9.1.1.1 This 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 DNV-OS-C502.

9.1.1.2 Design procedures for design of grouted connections with and without shear keys are given in [9.2] and [9.3]. Alternative methods for design of tubular grouted connections are given in GL Renewables Certification, “Certification of Grouted Connections for Offshore Wind Turbines”, Technical Note, Rev. 0, December 2013.

9.1.1.3 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 pile-to-sleeve or pile-to-structure grouted connections, single- or double-skin grouted tubular joints, and grout-filled tubes.

9.1.1.4 The principle for grouted connections in monopile structures is illustrated in Figure 9-1. 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 which is an extension of the tower.

Figure 9-1
Principle of grouted connection in monopile structure

9.1.1.5 Grouted connections in monopiles can 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 which are to transfer axial force shall always be designed and constructed with shear keys.

Guidance note:
Alternating bending moments implies 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.
9.1.1.6 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.

9.1.1.7 Types of grouted connections not specifically covered by this standard shall be specially considered.

9.1.1.8 All relevant factors which may influence the strength 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 modulus of elasticity
- resistance against wear
- tubular 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).

9.1.1.9 Grout materials shall comply with the requirements given in Sec.6 C “Grout Materials and Material Testing”.

9.1.1.10 The characteristic compression strength, $f_{cck}$, of the grout is defined as the 5% quantile in the probability distribution of the compression strength. The characteristic compression strength shall be estimated with at least 75% confidence. The characteristic grout compression strength, $f_{cck}$, shall be based on measurements on 150×300 mm cylinders. Smaller test specimens such as 75 mm cubes may be used for quality control provided the conversion factor between 150×300 mm cylinders and the actual test specimens are formally documented by tests for the actual grout. Note that the empirical formulas for characteristic interface shear capacity in [9.2.3.8], [9.2.4.4] and [9.2.5.4] are based on characteristic compressive strength on 75 mm cubes, $f_{ck}$.

9.1.1.11 The characteristic compression strength, $f_{cck}$, shall be converted to characteristic in-situ compression strength, $f_{cn}$, by means of the following formula:

$$f_{cn} = f_{cck} \cdot (1 - \frac{f_{cck}}{600})$$

where

$f_{cck} = \text{characteristic grout cylinder strength in units of MPa.}$

9.1.1.12 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. For further details regarding determination of $f_{tk}$ see DNV-OS-C502. 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. Unless test data indicate otherwise the conversion factor between the flexural tensile strength and the direct tensile strength may be taken as 0.40 for filled, pre-packed blended high performance grouts (with characteristic compressive strength in excess of 80 MPa). For neat cement grouts the tensile strength shall be documented by testing.

9.1.1.13 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:

$$f_{tn} = f_{tk} \cdot \left(1 - \left(\frac{f_{tk}}{25}\right)^{0.6}\right)$$

where

$f_{tk} = \text{characteristic direct tensile strength of the grout in units of MPa.}$

9.1.1.14 Grouted connections in wind turbine support structures shall be designed for the ULS and the FLS load combinations specified in Sec.5 for the loads specified in Sec.4.

9.1.1.15 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:

\[
\begin{align*}
    f_{cd} &= f_{cn} / \gamma_m \\
    f_{td} &= f_{tn} / \gamma_m
\end{align*}
\]

Requirements for the material factor \( \gamma_m \) are specified with each individual application in [9.2] and [9.3].

9.1.1.16 The characteristic value of the elastic modulus shall be taken as the mean value, and the dynamic modulus should be used.

9.1.2 Design principles

9.1.2.1 A grouted connection can be established with or without shear keys. An example with shear keys is given in Figure 9-1.

Guidance note:
Shear keys can 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.

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

9.1.2.2 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.

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

9.1.2.3 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 protect the grout more from wave action and associated wash-out during the curing of the grout. Full protection from wave action and associated wash-out during the curing of the grout will, however, require fitting of a protective cover of the exposed grout at the top of the grouted connection.

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

9.1.2.4 Steel tubes and shear keys shall be designed according to Sec.7.

9.1.2.5 Local buckling in steel tubes shall be considered.

9.1.2.6 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 [9.2] and [9.3].

9.2 Ultimate limit states

9.2.1 Tubular and conical grouted connections in monopiles without shear keys

9.2.1.1 Requirements for tubular and conical grouted connections in monopiles without shear keys are given in [9.2.1.2] through [9.2.1.8]. For tubular grouted connections it is a prerequisite that the connection shall not transfer axial force. 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 [9.2.2].

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

9.2.1.2 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.

9.2.1.3 Unless data indicate otherwise, the maximum nominal contact pressure $p_{nom}$ at the top and at the bottom of the grouted connection, caused by an applied bending moment $M$, can be found from the following expression

$$p_{nom} = \frac{3\pi M}{R_p^2 L_g (\pi + 3\mu) + 3\pi \mu R_p^2 L_g}$$

in which $\mu$ is the friction coefficient, $L_g = L - 2t_g$ 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 can be estimated as

$$p_{shear} = \frac{Q}{2R_p L_g}$$

where $Q$ denotes the shear force.

9.2.1.4 When the expression for the maximum nominal contact pressure in [9.2.1.3] is used to estimate the design value of the maximum nominal contact pressure for use in design, a characteristic value of the friction coefficient $\mu$ shall be used in conjunction with the design value of the applied bending moment $M$. The characteristic value of the friction coefficient $\mu$ 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.

9.2.1.5 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

$$p_{local} = SCF \cdot p_{nom}$$

where an analytical result for the stress concentration factor SCF reads

$$SCF = 1 + 0.025 \cdot \left(\frac{R}{t}\right)^{3/2} ; 2250 \text{ mm} \leq R \leq 3250 \text{ mm} \text{ and } 50 \text{ mm} \leq t \leq 100 \text{ mm}.$$ 

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 $t_g$ 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 $R_S$ and the wall thickness $t_S$ of the sleeve or transition piece, as applicable.

Alternatively, the local design contact pressure can be obtained from a linear finite element analysis. Such linear finite element analysis needs to be calibrated, see [9.2.7]. In this analysis, the grout material is to be modelled as linear, whereas contact elements can be nonlinear.

9.2.1.6 The design tensile stress in the grout shall be taken as

$$\sigma_d = 0.25 \cdot p_{local,d} \cdot \left(\sqrt{1 + 4 \cdot \mu_{local}^2} - 1\right)$$

where $\mu_{local}$ 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 $\mu_{local}$ shall be used, which in this case is defined as the mean value. Unless data indicate otherwise, the
characteristic value of \( \mu_{\text{local}} \) can be set equal to 0.7.

9.2.1.7 The design criterion against local fracture in tension is

\[
\sigma_u \leq \frac{f_m}{\gamma_m}
\]

The material factor shall be taken as \( \gamma_m = 1.5 \) and the criterion shall be satisfied at either end of the grouted connection.

9.2.1.8 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 to be transferred from the transition piece to the pile, this sliding resistance can be counted on in design if combined with regular inspections of rotation in operation. The torque capacity that can be counted on can then be taken as the capacity that results from the vertical load in combination with friction with a friction coefficient of 0.7.

9.2.2 Special provisions for conical grouted connections in monopiles without shear keys

9.2.2.1 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.

9.2.2.2 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.

9.2.2.3 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 can be expressed as

\[
M_e = \frac{2\pi}{\gamma_m} p \mu L_g (R_{pt}^2 + R_g L_g \sin \alpha + \frac{L_g^2}{3} \sin^3 \alpha)
\]

in which \( p_{\text{nom}} \) 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), \( L_g = L - 2 t_g \) 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 \( \alpha \) is the cone angle, i.e. the angle with the vertical axis of the cone. The friction coefficient \( \mu \) should be represented by its mean value. Unless data indicate otherwise, \( \mu \) can be set equal to 0.7. For design of a conical grouted connection against torque, the material factor \( \gamma_m \) shall be taken as 1.0, as there is some reserve capacity due to surface irregularities.

9.2.2.4 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 \( \delta_v \). For a vertical settlement \( \delta_v \), there will be a horizontal displacement \( \delta \),

\[
\delta = \delta_v \cdot \tan \alpha
\]

where \( \alpha \) is the cone angle measured from the vertical plane.

The pressure between steel and grout can be calculated as

\[
p_{\text{nom}} = \frac{E \cdot \delta}{F \cdot R_p}
\]
where the flexibility $F$ is

$$F = \left[ \frac{R_p}{t_p} + \frac{t_g E}{E_g R_p} + \frac{R_s}{t_s} \right]$$

in which $R_p$ is outer radius of pile and $R_s$ is outer radius of sleeve, both averaged over the height of the grouted connection. Further, $t_p$ is wall thickness of pile, $t_s$ is wall thickness of sleeve, $t_g$ is thickness of grout, $E$ is modulus of elasticity for steel and $E_g$ is modulus of elasticity 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 $\mu$ should be represented by its characteristic value. Unless data indicate otherwise, $\mu$ can be set equal to 0.7. 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

$$A_{cone} = 2 \pi L_g R_{pl} + \pi L_g^2 \sin \alpha$$

When the pressure $p_{nom}$ is known, the resulting displacement $\delta$ and the resulting settlement $\delta_v$ can be calculated from the above expressions.

9.2.2.5 The available friction capacity cannot be utilized fully to carry the applied torque according to [9.2.2.3] and at the same time also be utilized fully to carry the applied dead load according to [9.2.2.4]. 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.

9.2.3 Tubular grouted connections in monopiles with shear keys

9.2.3.1 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.

9.2.3.2 The shear keys shall be placed in the central region of the grouted connection as indicated in Figure 9-2, 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 centred in the grouted connection midpoint and has a distance $L_g/4$ to either end of the effective part of the grout.

The following assumptions are prerequisites for the design procedure in [9.2.3.3] to [9.2.3.16]:

— 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 9-2
— The number of shear keys on the TP and the number of shear keys of the pile differ by one
— The grout layer should not be disturbed by any other member of design.
9.2.3.3 Unless data indicate otherwise, the maximum nominal radial contact pressure $p_{\text{nom}}$ at the top and at the bottom of the grouted connection, caused by an applied bending moment $M$, shall be derived from the following expression:

$$
p_{\text{nom}} = \frac{3\pi M \cdot EL_g}{EL_g \cdot \left\{ \frac{R_p}{L_g} \left( \pi + 3\mu \right) + 3\pi \mu R_p^2 L_g \right\} + 18\pi^2 k_{\text{eff}} R_p^3 \left( \frac{R_p^2}{t_p} + \frac{R_{\text{TP}}^2}{t_{\text{TP}}} \right)}
$$

in which:

- $k_{\text{eff}}$ = effective spring stiffness for the shear keys
- $\mu$ = characteristic friction coefficient, equal to 0.7
- $R_p$ = outer radius of the pile
- $R_{\text{TP}}$ = outer radius of transition piece
- $t_p$ = wall thickness of pile
- $t_{\text{TP}}$ = wall thickness of transition piece
- $L_g = L - 2t_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

![Diagram](image-url)
For design against the ULS, the applied bending moment \( M \) shall be taken as the design bending moment including load factors as specified in Sec.5. For design against the FLS, \( M \) shall be taken as the maximum bending moment caused by dynamic loads multiplied by a load factor of 1.0.

9.2.3.4 The effective spring stiffness per unit length around the circumference of the grouted connection for \( n \) shear keys is expressed as

\[
k_{\text{eff}} = \frac{2 t_{TP} s_{\text{eff}}^2 n E \psi}{4 \sqrt{3(1-\nu^2)} t_g^2 \left( \frac{R_p}{t_p} \right)^{3/2} + \left( \frac{R_{TP}}{t_{TP}} \right)^{3/2} \left( t_{TP} + n s_{\text{eff}}^2 L_g \right)}
\]

in which

- \( s_{\text{eff}} \) = 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 of steel = \( 2.1 \times 10^5 \) MPa
- \( \nu \) = Poisson’s ratio for steel = 0.3
- \( n \) = number of effective shear keys (the actual number of shear keys on each side of the grouted connection is \( n+1 \))
- \( \psi \) = design coefficient
  - 1.0 for calculation of load action on shear keys
  - 0.5 for calculation of maximum nominal radial contact pressure

9.2.3.5 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

\[
F_{V_{Shk}} = \frac{6 p_{\text{nom}} k_{\text{eff}} R_p}{E} \left( \frac{R_p}{t_p} \right) \frac{R_{TP}^2}{t_{TP}} + \frac{P}{2 \pi R_p}
\]

where

- \( P \) = self-weight of structure above the pile, including full weight of the transition piece.

9.2.3.6 The average action force per unit length along the circumference on one shear key shall be taken as

\[
F_{V1_{Shk}} = \frac{F_{V_{Shk}}}{n}
\]

9.2.3.7 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 \( \varphi \) is the installation angle tolerance with respect to verticality.

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

\[
F_{V_{Shk,mod}} = \frac{6 p_{\text{nom}} k_{\text{eff,mod}} R_p}{E} \left( \frac{R_p}{t_p} \right) \frac{R_{TP}^2}{t_{TP}}
\]

in which
\[ k_{\text{eff,mod}} = \frac{2 t_{TP} s_{\text{eff}}^2 n E \psi}{4 \sqrt[3]{3(1-\nu^2)} t_{g,\text{min}}^2 \left( \frac{R_p}{t_p} \right)^{3/2} + \left( \frac{R_{TP}}{t_{TP}} \right)^{3/2}} t_{TP} + n s_{\text{eff}}^2 L_g \]

is a modified effective spring stiffness for the loading on the \( n \) shear keys, based on the minimum grout thickness \( t_{g,\text{min}} \) calculated as

\[ t_{g,\text{min}} = t_g - \Delta t_g \]

The average action force per unit length along the circumference on one shear key shall be taken as

\[ F_{V1,\text{Shk,mod}} = \frac{F_{V \text{Shk,mod}}}{n} \]

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.

9.2.3.8 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

\[ k = \left[ \frac{2 R_p}{t_p} + \frac{2 R_{TP}}{t_{TP}} \right]^{-1} + \left( \frac{E_g}{E} \right) \left[ \frac{2 R_{TP} - 2 t_{TP}}{t_g} \right]^{-1} \]

- \( E_g \) = Young’s modulus of the grout
- \( f_{ck} \) = 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 9-2 and Figure 9-4 for explanation of symbols.

9.2.3.9 The characteristic capacity per unit length of one shear key is

\[ F_{V1,\text{Shk,\text{cap}}} = f_{bk} s \]

The design capacity per unit length of one shear key is

\[ F_{V1,\text{Shk,\text{cap,d}}} = \frac{F_{V1,\text{Shk,\text{cap}}}}{\gamma_m} \]

The material factor shall be taken as \( \gamma_m = 2.0 \).
9.2.3.10 The design criterion for the grouted connection with shear keys is

\[ F_{V1Shk} \leq F_{V1Shk \ cap,d} \]

For design of the shear keys, reference is made to Sec.7.

9.2.3.11 The following requirement for the vertical distance between shear keys shall be fulfilled:

\[ s \geq \min \left\{ \frac{0.8 \sqrt{R_p t_p}}{0.8 \sqrt{R_{tp} t_{tp}}} \right\} \]

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 still be carried out on the basis of the specified requirement for the vertical distance between the shear keys.

9.2.3.12 The following requirements for the geometry of the shear keys shall be fulfilled:

\[ 1.5 \leq \frac{w}{h} \leq 3.0 \quad \text{and} \quad \frac{h}{s} \leq 0.10 \]

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.

9.2.3.13 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

- \( L_g = L - 2t_g \) = 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.

9.2.3.14 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.

9.2.3.15 The following requirement for the geometry of the transition piece shall be fulfilled:

\[ 9 \leq \frac{R_{TP}}{t_{TP}} \leq 70 \]

9.2.3.16 The following requirement for the maximum nominal radial contact pressure shall be fulfilled:

\[ p_{nom} \leq 1.5 \text{ MPa} \]

This requirement for the nominal radial contact pressure can be waived if a detailed FE analysis is performed and a fatigue assessment of the grouted part is documented according to requirements for fatigue design of concrete in Sec.8.
Guidance note:
The effective spring stiffness to be applied for calculation of the nominal contact pressure $p_{\text{nom}}$ requires use of a design coefficient $\psi = 0.5$, see [9.2.3.4].

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

9.2.3.17 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 [9.2.3.2]. The torsional resistance resulting from permanent weight of the structure above the transition piece, and including the transition piece, can be counted on when the need for vertical shear keys is assessed.

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 9-3.

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

![Figure 9-3](image)

Examples of vertical shear key layouts for torsional capacity

9.2.3.18 The design load per unit length of one vertical shear key is

$$F_{H1Shk} = \frac{M_T}{R_p \cdot L_S \cdot n}$$

in which $M_T$ 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.

9.2.3.19 The characteristic interface shear capacity for torque in the grouted connection with vertical shear keys is

$$f_{hk} = \left[ \frac{800}{D_p} + 140 \left( \frac{h}{s} \right)^{0.8} \right]^{-0.6} f_{ck}^{0.3}$$

in which the parameters are the same as those specified in [9.2.3.8], except for the parameter $s$ which is here the horizontal distance between vertical shear keys, measured along the circumference of the monopile. Units are as specified in [9.2.3.8]. 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,

$$f_{hk} = \left[ 0.75 - 1.4 \left( \frac{h}{s} \right) \right] f_{ck}^{0.5}$$
9.2.3.20 The characteristic capacity per unit length of one shear key is

\[ F_{H1\text{ Shk\, cap}} = f_{hk} s \]

9.2.3.21 The design capacity per unit length of one shear key is

\[ F_{H1\text{ Shk\, cap,d}} = \frac{F_{H1\text{ Shk\, cap}}}{\gamma_m} \]

The material factor shall be taken as \( \gamma_m = 2.0 \).

9.2.3.22 The design criterion for the vertical shear keys is

\[ F_{H1\text{ Shk}} \leq F_{H1\text{ Shk\, cap,d}} \]

9.2.4 Tubular grouted connections with shear keys in jacket structures with post-installed piles

9.2.4.1 Provisions for tubular grouted connections with shear keys in jacket structures with post-installed piles are given in [9.2.4.2] through [9.2.4.15]. The provisions apply to tubular connections between jacket structures and conventional piles, where the piles are post-installed within pile sleeves or through jacket legs, for example by driving or by drilling and grouting.

9.2.4.2 For design of tubular grouted connections in jacket structures, it is important to avoid 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 [9.3.2] is followed.

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

9.2.4.3 The design load per unit length along the circumference of one shear key shall be taken as

\[ F_{V1\text{ Shk}} = \frac{P_{a,d}}{2\pi \cdot R_p \cdot n} \]

where \( R_p \) is the outer radius of the pile, \( P_{a,d} \) 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.

9.2.4.4 The interface shear capacity in the grouted connection with shear keys shall be taken as

\[ f_{hk} = \left[ \frac{800}{D_p} + 140 \left( \frac{h}{s} \right)^{0.8} \right] k^{0.6} f_{ck}^{0.3} \]

where
- \( h \) = height of shear key measured radially from the grout-steel interface in units of mm
- \( D_p \) = pile diameter in units of mm
- \( f_{ck} \) = characteristic compressive strength of 75 mm cubes in units of MPa
- \( s \) = vertical centre-to-centre distance between shear keys in units of mm
- \( k \) = radial stiffness parameter defined as
$$k = \left[ \frac{2R_p}{t_p} + \left( \frac{2R_s}{t_s} \right) \right]^{-1} + \left( \frac{E_g}{E} \right) \left[ \frac{2R_s - 2t_s}{t_s} \right]^{-1}$$

$E_g$ = Young's modulus of the grout
$R_s$ = outer radius of sleeve
$t_s$ = wall thickness of sleeve

However, the interface shear capacity shall not be taken larger than the limit set forth by grout matrix failure,

$$f_{bk} = \left[ 0.75 - 1.4 \left( \frac{h}{s} \right) \right] f_{ck}^{0.5}$$

Reference is made to Figure 9-4 for explanation of symbols.

**Figure 9-4**
Grouted pile-to-sleeve connection in jacket structure

9.2.4.5 The characteristic capacity per unit length of one shear key is

$$F_{V1Shk \text{ cap}} = f_{bk} \cdot s$$

The design capacity per unit length of one shear key is

$$F_{V1Shk \text{ cap,d}} = \frac{F_{V1Shk \text{ cap}}}{\gamma_m}$$

The material factor shall be taken as $\gamma_m = 2.0$.

9.2.4.6 The design criterion for the grouted connection with shear keys is

$$F_{V1Shk} \leq F_{V1Shk \text{ cap,d}}$$

For design of the shear keys, reference is made to Sec.7.
9.2.4.7 The following requirement for the vertical distance between shear keys shall be fulfilled:

\[ s \geq \min \left\{ \frac{0.8}{0.8, \sqrt{R_p t_p}} \right\} \]

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.

9.2.4.8 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 \( h/D_p \leq 0.012 \) is fulfilled, where \( D_p \) denotes the outer pile diameter.

9.2.4.9 It is recommended that the grout-length-to-pile-diameter ratio is kept within the following range:

\[ 1 \leq \frac{L_g}{D_p} \leq 10 \]

in which \( L_g \) denotes the effective length of the grouted section and \( D_p \) denotes the outer pile diameter. When \( L_g/D_p < 2.5 \), then the design procedure in [9.2.3] for grouted connections in monopiles can be used as an alternative for documentation of grouted connections in jacket structures with post-installed piles.

9.2.4.10 It is recommended that the grout dimensions meet the following limitations:

\[ 10 \leq \frac{D_g}{t_g} \leq 45 \]

in which \( D_g \) denotes the outer diameter of the grout and \( t_g \) denotes the nominal grout thickness.

9.2.4.11 The following requirement for the geometry of the pile shall be fulfilled:

\[ 10 \leq \frac{R_p}{t_p} \leq 30 \]

9.2.4.12 The following requirement for the geometry of the sleeve shall be fulfilled:

\[ 15 \leq \frac{R_s}{t_s} \leq 70 \]

9.2.4.13 For connections involving post-installed piles, the region which is not 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 upwards. The elastic length of the pile can be taken as

\[ l_e = \frac{4EI_p}{k_{rD}} \]

where \( I_p \) = moment of inertia of the pile.
The supporting spring stiffness $k_{rD}$, defined as the radial spring stiffness times the pile diameter, can be expressed as

$$k_{rD} = \frac{4EI_p}{R_p^2 + \left(\frac{R_p}{t_p} + \frac{R_s}{t_s} + t_g \cdot m\right)}$$

where

- $R_p = \text{radius to outer part of pile}$
- $t_p = \text{thickness of pile}$
- $R_s = \text{radius to outer part of sleeve}$
- $t_s = \text{thickness of sleeve}$
- $E = \text{Young’s modulus for steel}$
- $m = \text{ratio of Young’s modulus for steel and Young’s modulus for grout material}; m = 18 \text{ can be used if Young’s modulus for grout material is not known.}$

For connections involving post-installed piles, the region significantly affected by the bending moment is the region from a level half an elastic length 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.

9.2.4.14 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

$$p_{nom} = \frac{l_c^2 \cdot k_{rD}}{8EI_p R_p} \cdot (M_0 + Q_0 \cdot l_c)$$

in which

- $l_c = \text{elastic length of pile as defined in [9.2.4.13]}$
- $k_{rD} = \text{support spring stiffness as defined in [9.2.4.13]}$
- $E = \text{Young’s modulus of steel} = 2.1 \cdot 10^5 \text{ MPa.}$
- $I_p = \text{moment of inertia of the pile}$
- $R_p = \text{outer radius of the pile}$

9.2.4.15 The following requirement for the nominal contact pressure due to bending and transverse shear force shall be fulfilled:

$$p_{nom} \leq 1.5 \text{ MPa}$$

This requirement for the nominal contact pressure due to bending and transverse shear force can be waived if a detailed FE analysis is performed and a fatigue assessment of the grouted part is documented according to requirements for fatigue design of concrete in Sec. 8.

9.2.5 Tubular grouted connections with shear keys in jacket structures with preinstalled piles

9.2.5.1 Provisions for tubular grouted connections with shear keys in jacket structures with preinstalled piles are given in [9.2.5.2] through [9.2.5.15]. The provisions apply to tubular connections which are formed such that a jacket leg stub is fitted inside the top of a preinstalled pile.

9.2.5.2 For design of tubular grouted connections in jacket structures, it is important to avoid 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 must 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 [9.3.2] is followed.

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

9.2.5.3 The design load per unit length along the circumference of one shear key shall be taken as

\[ F_{V1Shk} = \frac{P_{a,d}}{2\pi R_{JL} \cdot n} \]

where \( R_{JL} \) is the outer radius of the jacket leg, \( P_{a,d} \) 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.

9.2.5.4 The interface shear capacity in the grouted connection with shear keys shall be taken as

\[ f_{hk} = \left[ \frac{800}{D_{JL}} + 140 \left( \frac{h}{s} \right)^{0.8} \right] k^{0.6} f_{ck}^{0.3} \]

where

- \( h \) = height of shear key measured radially from the grout-steel interface
- \( D_{JL} \) = jacket leg diameter in units of mm
- \( f_{ck} \) = 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

\[ k = \left[ (2R_{JL}/t_{JL}) + (2R_p/t_p) \right]^{-1} + (E_g/E) \left[ (2R_p - 2t_p)/t_g \right]^{-1} \]

- \( E_g \) = Young’s modulus of the grout
- \( R_p \) = outer radius of pile
- \( t_p \) = wall thickness of pile

However, the interface shear capacity shall not be taken larger than the limit set forth by grout matrix failure,

\[ f_{hk} = \left[ 0.75 - 1.4 \left( \frac{h}{s} \right) \right] f_{ck}^{0.5} \]

Reference is made to Figure 9-5 for explanation of symbols.
9.2.5.5 The characteristic capacity per unit length of one shear key is

$$F_{V1,Shk\,cap} = f_{sk}\,s$$

The design capacity per unit length of one shear key is

$$F_{V1,Shk\,cap,d} = \frac{F_{V1,Shk\,cap}}{\gamma_m}$$

The material factor shall be taken as $\gamma_m = 2.0$.

9.2.5.6 The design criterion for the grouted connection with shear keys is

$$F_{V1,Shk} \leq F_{V1,Shk\,cap,d}$$

For design of the shear keys, reference is made to Sec.7.

9.2.5.7 The following requirement for the vertical distance between shear keys shall be fulfilled:

$$s \geq \min \left\{ \frac{0.8}{R_{IL}} \frac{t_{IL}}{L}, \frac{0.8}{R_p} \frac{t_p}{L} \right\}$$

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.

9.2.5.8 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 $h/D_{JL} \leq 0.012$ is fulfilled, where $D_{JL}$ denotes the outer jacket leg diameter.

**9.2.5.9** It is recommended that the grout-length-to-leg-diameter ratio is kept within the following range:

$$1 \leq \frac{L_g}{D_{JL}} \leq 10$$

in which $L_g$ denotes the effective length of the grouted section and $D_P$ denotes the outer pile diameter. When $L_g/D_{JL} < 2.5$, then the design procedure in [9.2.3] for grouted connections in monopiles can be used as an alternative for documentation of grouted connections in jacket structures with preinstalled piles.

**9.2.5.10** It is recommended that the grout dimensions meet the following limitations:

$$10 \leq \frac{D_g}{t_g} \leq 45$$

in which $D_g$ denotes the outer diameter of the grout and $t_g$ denotes the nominal grout thickness.

**9.2.5.11** The following requirement for the geometry of the jacket leg shall be fulfilled:

$$10 \leq \frac{R_{JL}}{t_{JL}} \leq 30$$

**9.2.5.12** The following requirement for the geometry of the pile shall be fulfilled:

$$15 \leq \frac{R_p}{t_p} \leq 70$$

**9.2.5.13** For connections involving preinstalled piles, the region which is not 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 downwards.

The elastic length of the pile can be taken as

$$l_e = \sqrt{\frac{4EI_{JL}}{k_{rD}}}$$

where $I_{JL}$ = moment of inertia of the jacket leg.

The supporting spring stiffness $k_{rD}$, defined as the radial spring stiffness times the jacket leg diameter, can be expressed as

$$k_{rD} = \frac{4ER_{JL}}{\left(\frac{R_{JL}^2}{t_{JL}} + \frac{R_p^2}{t_p} + t_g\right)m}$$

where

- $R_{JL}$ = radius to outer part of jacket leg
- $t_{JL}$ = thickness of jacket leg
- $R_p$ = radius to outer part of pile
- $t_p$ = thickness of pile
- $E$ = Young’s modulus for steel
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.

For connections involving preinstalled piles, the region 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. 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.

9.2.5.14 Unless data indicate otherwise, the maximum nominal radial contact pressure \( p_{\text{nom}} \) at the grouted connection, caused by the design horizontal shear force \( Q_0 \) and the design bending moment \( M_0 \) at the top of the pile in case of preinstalled piles, shall be derived from the following expression

\[
p_{\text{nom}} = \frac{l_e^2 \cdot k_{D} \cdot (M_0 + Q_0 \cdot l_e)}{8EI_{JL}R_{JL}}
\]

in which

\( l_e \) = elastic length of pile as defined in [9.2.5.13]
\( k_{D} \) = support spring stiffness as defined in [9.2.5.13]
\( E \) = Young’s modulus of steel = 2.1·10\(^5\) MPa.
\( I_{JL} \) = moment of inertia of the jacket leg
\( R_{JL} \) = outer radius of the jacket leg

Jackets with preinstalled piles often have large tolerances. When such large tolerances are in place, the moment of eccentricity associated with the tolerances should be included in design in addition to the design bending moment \( M_0 \).

9.2.5.15 The following requirement for the nominal contact pressure due to bending and transverse shear force shall be fulfilled:

\[ p_{\text{nom}} \leq 1.5 \text{ MPa} \]

This requirement for the nominal contact pressure due to bending and transverse shear force can be waived if a detailed FE analysis is performed and a fatigue assessment of the grouted part is documented according to requirements for fatigue design of concrete in Sec.8.

9.2.6 Abrasive wear of contact surfaces

9.2.6.1 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. 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 \( \gamma_f \) equal to 1.0 can be used.

9.2.6.2 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 in-service experience exists. It is also recommended to perform calculations of the resulting abrasive wear.

9.2.7 Finite element analysis

9.2.7.1 Finite element analysis (FEM) can be used for assessment of the ultimate capacity of grouted connections. Guidance for finite element analysis is given in App.K.

9.2.7.2 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, it is recommended that the analysis methodology is calibrated to capacities for well-known cases or to reliable test data wherever such data exist.

9.2.7.3 The analytical expressions for contact pressure in this section can be used as basis for calibration of the finite element analysis methodology in absence of other reliable data. The calibration analysis should 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.
9.3 Fatigue limit states

9.3.1 Conical grouted connections in monopiles without shear keys

9.3.1.1 All 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.

9.3.1.2 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 shall be documented.

9.3.1.3 The effects of significant dynamic response shall be properly accounted for when stress ranges are determined. Special care is to be taken to adequately determine the stress ranges in structures or members excited in the resonance range. The amount of damping assumed is to be appropriate to the design.

9.3.1.4 The geometrical layout of the structural elements and reinforcement, if required, is to be such as to minimize the possibility of fatigue failure.

9.3.1.5 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.

9.3.1.6 For structures subjected to multiple stress cycles, it shall be demonstrated that the structure will endure the expected stresses during the required design life.

9.3.1.7 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.

9.3.1.8 The design fatigue factor DFF shall be taken as 3.0.

9.3.1.9 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 is to 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.

9.3.1.10 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.

9.3.1.11 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 9.3.1.13.

9.3.1.12 When the characteristic S-N curve is to be estimated from test data, it shall be estimated with at least 75% confidence.

9.3.1.13 Unless data indicate otherwise, the characteristic number of cycles to failure, $N$, of grout subjected to cyclic stresses can be calculated from

$$\log_{10} N = C_1 \cdot \left(1 - \frac{\sigma_{\text{max}}}{f_{rd}}\right) / \left(1 - \frac{\sigma_{\text{min}}}{f_{rd}}\right)$$

where $f_{rd} =$ design reference strength, $\sigma_{\text{max}}$ is the largest value of the maximum principal compressive stress during a stress cycle within the stress block and $\sigma_{\text{min}}$ 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, $\sigma_{\text{min}}$ 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 $\sigma_{\text{max}}$ and $\sigma_{\text{min}}$ shall be set equal to zero when the design number of cycles to failure is calculated.

The factor $C_1$ is to 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

$$ X = C_1/(1 - \frac{\sigma_{\text{min}}}{f_{\text{rd}}} + 0.1 \cdot C_1) $$

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

$$ C_2 = (1 + 0.2 \cdot (\log_{10} N - X)) $$

The design reference strength $f_{\text{rd}}$ is calculated as

$$ f_{\text{rd}} = C_5 \cdot \frac{f_{\text{cm}}}{\gamma_m} $$

in which

- $f_{\text{cm}}$ is the nominal compression strength, see [9.1.1.7]
- $\gamma_m$ is the material factor, to be taken according to Table 9-1
- $C_5$ 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 DNV-OS-C502 Sec.6M.

The grout thickness limit of 150 mm used to distinguish between types of structural connections in Table 9-1 refers to nominal grout thickness.

Guidance note:
The fatigue adjustment factor $C_5$ can take on values above as well as below 1.0, depending on the grout material.

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

### Table 9-1 Requirements for material factor $\gamma_m$ for grout in the FLS

<table>
<thead>
<tr>
<th>Type of structural connection</th>
<th>Material factor $\gamma_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type A steel tubular grouted joints (Grout thickness less than 150 mm)</td>
<td>1.5</td>
</tr>
<tr>
<td>Type B steel tubular grouted joints (Grout thickness greater than 150 mm)</td>
<td>1.75</td>
</tr>
<tr>
<td>Type C steel tubular grouted joints (Grout reinforced by reinforcement bars)</td>
<td>1.35</td>
</tr>
</tbody>
</table>

9.3.1.14 Design of grouted connections against fatigue failure according to the procedure and the requirements given in [9.3.1.1] through [9.3.1.13] will meet the requirements for fatigue design set forth in IEC 61400-1.

9.3.2 Tubular grouted connections with shear keys, general

9.3.2.1 All load 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 load amplitudes is determined.

9.3.2.2 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 shall be documented.

9.3.2.3 The effects of significant dynamic response shall be properly accounted for when load amplitudes are determined. Special care is to be taken to adequately determine the load amplitudes in structures or members excited in the resonance range. The amount of damping assumed is to be appropriate to the design.

9.3.2.4 The geometrical layout of the structural elements and the shear keys is to be such as to minimize the possibility of fatigue failure.

9.3.2.5 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.

9.3.2.6 For structures subjected to multiple stress cycles, it shall be demonstrated that the structure will endure the expected stresses during the required design life.

9.3.2.7 Vertical downward dynamic loading on a shear key will potentially lead to cracking along compression struts just below the shear key. Vertical upward dynamic loading on a shear key will potentially lead to cracking along compression struts just above the shear key. To design the grouted connection against fatigue failure in both zones, two long term distributions of stress amplitudes shall be established:

- Long term distribution of amplitude of vertical downward dynamic load on shear key. The distribution consists of one downward amplitude value per load cycle, see Figure 9-6.
- Long term distribution of amplitude of vertical upward dynamic load on shear key. The distribution consists of one upward amplitude value per load cycle, see Figure 9-6.

Guidance note:
Each of the two long term amplitude distributions contains contributions from load cycles occurring for different horizontal directions of the environmental loading. The static load level, about which the dynamic cyclic loading takes place and from which the amplitude is measured, may be different from one horizontal direction of the environmental loading to another.

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

![Figure 9-6](image)

**Figure 9-6**
Load transfer between shear key and grout during moment loading of grouted connection

9.3.3 Tubular grouted connections in monopiles with shear keys

9.3.3.1 The following requirement for vertical distance between shear keys shall be fulfilled

\[ s \geq \min \left\{ 0.4 \sqrt{R_p t_p} \right\} \]

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

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

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---
9.3.3.2 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

\[
\begin{align*}
\log N &= 5.400 - 8y \quad \text{for } y \geq 0.30 \\
\log N &= 7.286 - 14.286y \quad \text{for } 0.16 < y < 0.30 \\
\log N &= 13.000 - 50y \quad \text{for } y \leq 0.16
\end{align*}
\]

in which, for each load cycle, the relative load level is

\[
y = \frac{F_{V1\text{ Shk}} \gamma_m}{F_{V1\text{ Shk cap}}}
\]

where \( F_{V1\text{ Shk cap}} \) is the characteristic shear key capacity as described in [9.2.3.9], \( F_{V1\text{ Shk}} \) is the shear key load of the load cycle in question, consisting of the static load plus the load amplitude of the load cycle in question, and \( \gamma_m \) is the material factor. The material factor shall be taken as \( \gamma_m = 1.5 \) when this approach to fatigue design is applied.

9.3.3.3 When the expression in [9.2.3.5] is applied for calculation of loads to be used in the fatigue evaluation, the load amplitude can be replaced by half the load range in cases where the downward load on the shear key due to the self-weight \( P \) is larger than the reversed upward load on the shear key from the variable bending moment and the variable axial force.

9.3.3.4 The design criterion is

\[
D = \sum_{j=1}^{k} \sum_{i=1}^{n_0} \frac{n_i}{N_i} \leq 1.0
\]

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

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.

9.3.3.5 The effect of installation tolerances on the fatigue capacity should be assessed. Reference is made to [9.2.3.7].

9.3.4 Tubular grouted connections with shear keys in jacket structures

9.3.4.1 Provisions for fatigue design of tubular grouted connections with shear keys in jacket structures are given in [9.3.4.2] to [9.3.4.5]. 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.

9.3.4.2 The following requirement for vertical distance between shear keys in jacket structures with post-installed piles shall be fulfilled

\[
s \geq \min \left\{ 0.4 \sqrt{R_p t_p}, 0.4 \sqrt{R_s t_s} \right\}
\]

The following requirement for vertical distance between shear keys in jacket structures with preinstalled piles shall be fulfilled
9.3.4.3 Design against the FLS for grouted connections 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 stress level $y$, is represented as

$$ s \geq \min \left\{ \frac{0.4 \sqrt{R_{yI} t_{pl}}}{0.4 \sqrt{R_{yF} t_{pl}}} \right\} $$

in which, for each stress cycle, the relative stress is

$$ y = \frac{F_{V1Shk} Y_m}{F_{V1Shk cap}} $$

where $F_{V1Shk cap}$ is the characteristic shear key capacity as described in [9.2.4.5], $F_{V1Shk}$ is the shear key load of the load cycle in question, consisting of the static load plus the load amplitude of the load cycle in question, and $Y_m$ is the material factor. The material factor shall be taken as $Y_m=1.5$ when this approach to fatigue design is applied.

9.3.4.4 When the long term history of load amplitudes to be used for fatigue design has been established according to the specifications in [9.3.2], the load amplitudes of the individual load cycles can be replaced by half the load range for those load cycles where the permanent static axial load due to deadweight is greater than the load range.

9.3.4.5 The design criterion is

$$ D = \sum_{j=1}^{k} \sum_{i=1}^{n_j} \frac{N_i}{n_0} \leq 1.0 $$

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.

The design criterion shall be fulfilled for the following two cases:

- The cumulative damage $D$ is calculated based on the permanent static load and 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 permanent static load and the long term distribution of the amplitude of the vertical upward dynamic load on the shear key.

9.3.5 Shear keys

For fatigue assessment of shear keys, the S-N curve designated as Curve E in [7.10.2] Table 7-14 applies.

9.4 Grouting operations

9.4.1 General

9.4.1.1 The grouting operations of connections are to comply with relevant requirements given in DNV-OS-C502, Sec.7, Sec.9 and Appendix H.

9.4.1.2 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.

9.4.1.3 Injection of grout shall be carried out from the bottom of the annulus. Complete filling of the annulus
shall be confirmed by grout overfill of good quality at the top of the grouted connection or at the top outlet hole.

9.4.1.4 Sufficient strength of formwork or similar (e.g. an inflatable rubber seal) shall be ensured.

9.4.1.5 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.

9.4.1.6 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.

9.4.1.7 Adequate backup equipment for the grouting process shall be available before the grouting operation is initiated.

9.4.1.8 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.

9.4.1.9 In general, piling operations are not to be performed after commencement of pile-grouting operations.

9.4.2 Operations prior to grouting

All steel surfaces to be in contact with grout shall be clean before grouting. Before positioning of the tubes, the surfaces must be checked for grease, oil, paint, marine growth etc. and cleaned if necessary.

9.4.3 Monitoring

9.4.3.1 Parameters considered as important for controlling the grouting operation are to be monitored prior to and during the grouting operation. Records are to be kept of all monitored parameters. These parameters are normally to include:

— results from qualification tests for grout mix
— results from grout tests during operation
— records of grout density and flowability 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.

9.4.3.2 Means are to be provided for observing the emergence of grout from the evacuation point from the compartment or annulus being grouted.

9.4.4 Early age cycling

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 foundation pile-to-sleeve connections, the determination shall be based on an on-bottom analysis of the structure with ungrouted piles. If the expected relative axial movement at the grout-steel interface exceeds 1 mm during the 24-hour period, the movement shall be limited to maximum 1 mm by implementing suitable means. This limit on the relative movement between the grouted steel members also applies to grouted connections in monopile structures.
SECTION 10 FOUNDATION DESIGN

10.1 Introduction

10.1.1 General

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

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

10.1.1.3 Design of foundations shall be based on site-specific information, see Sec. 3.

10.1.1.4 The geotechnical design of foundations shall consider both the strength and the deformations of the foundation structure and of the foundation soils.

This section states requirements for

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

Requirements for the foundation structure itself are given in Sec.7 to Sec.9 as relevant for a foundation structure constructed from steel and/or concrete.

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

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

10.1.1.6 The definition of limit state categories as given in Sec.2 is valid for foundation design with the exception that failure due to effect of cyclic loading is treated as an ultimate limit state (ULS), alternatively as an accidental limit state (ALS), using partial load and material factors as defined for these limit state categories. The load factors are in this case to be applied to all cyclic loads in the design load history. Lower load factors than prescribed in Sec.5 may be accepted if the total safety level can be demonstrated to be within acceptable limits.

10.1.1.7 The load factors to be used for design related to the different categories of limit states are given in Sec.5.

10.1.1.8 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 [10.3].

10.1.1.9 Material factors shall be applied to soil shear strength as follows:

— for effective stress analysis, the tangent to the characteristic friction angle shall be divided by the material factor $\gamma_m$
— for total stress analysis, the characteristic undrained shear strength shall be divided by the material factor $\gamma_m$.

For soil resistance to axial pile load, material factors shall be applied to the characteristic resistance as described in [10.3.1.7].

For soil resistance to lateral pile load, material factors shall be applied to the characteristic resistance as described in [10.3.1.6].

10.1.1.10 Settlements caused by increased stresses in the soil due to structural weight shall be considered for structures with gravity type foundations. The risk of uneven settlements should be considered in relation to the tolerable tilt of the wind turbine support structure.

10.1.1.11 Further elaborations on design principles and examples of design solutions for foundation design are given in DNV Classification Notes 30.4.

10.1.2 Soil investigations

Requirements to soil investigations as a basis for establishing necessary soil data for a detailed design are given in Sec.3.
10.1.3 Characteristic properties of soil

10.1.3.1 The characteristic value of a soil property is defined as the 5% quantile in the distribution of the soil property, when a local value of the soil property governs the design and when a low value of the soil property is unfavourable for the design. When a high value of the soil property is unfavourable for the design, the 95% quantile shall be used as characteristic value instead of the 5% quantile. When the average of the soil property governs the design, the characteristic value shall be taken as the mean value of the soil property.

The characteristic value of a soil property shall account for the variability in that property based on an assessment of the soil volume that governs the limit state in consideration.

Guidance note:
Variability in a soil property is usually a variability of that property from point to point within a soil volume. When small soil volumes are involved, it is necessary to base calculations on the local soil property with its full variability. When large soil volumes are involved, the effect of spatial averaging of the fluctuations in the soil property from point to point over the soil volume comes into play. Calculations may then be based on the spatially averaged soil property, which eventually becomes equal to the mean of the soil property when the soil volume is large enough.

A couple of examples are given in the following: For axial pile capacity of a long pile in clay it is usually the average undrained shear strength along the pile which is of interest for design since local fluctuations of the strength from point to point along the pile can be assumed to average out over the length of the pile; hence the mean value of the strength can be used as characteristic value. Small footings a few metres wide have such a small extent and involve such a small soil volume that the soil strength used to calculate their capacity in practice is a local strength; hence an unfavourable low value is used as characteristic value.

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

10.1.3.3 The results of both laboratory tests and in-situ tests shall be evaluated and corrected as relevant on the basis of recognised practice and experience. Such evaluations and corrections shall be documented. In this process account shall be given to possible differences between properties measured in the tests and 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.

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

10.1.3.5 When the characteristic value of a soil property is estimated from limited data, the estimate shall be a cautious estimate of the value that affects the occurrence of the limit state. This implies that the characteristic value shall be estimated with confidence. It is recommended to apply a confidence of at least 75%.

Guidance note:
Relevant statistical methods should be used for estimation of characteristic values of soil properties.

For estimation of characteristic values of soil properties by means of statistical methods, reference is made to DNV-RP-C207.

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

10.1.4 Effects of cyclic loading

10.1.4.1 The effects of cyclic loading on the soil properties shall be considered in foundation design for offshore wind turbine structures.

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

10.1.4.3 In the SLS design condition the effects of cyclic loading on the soil’s shear modulus shall be corrected for as relevant when dynamic motions, settlements and permanent (long-term) horizontal displacements shall
be calculated. See also [10.4.5].

10.1.4.4 The effects of wave- and wind-induced forces on the soil properties shall be investigated for single storms, for normal operating conditions followed by a storm or an emergency shutdown, for several succeeding storms, and for any other wave and wind load condition that may influence the soil properties.

**Guidance note:**

Cyclic degradation of soil properties is the issue here, as well as rate effects.

---end---of---Guidance---note---

10.1.4.5 In seismically active areas, where the structure-foundation system may be subjected to earthquake forces, the deteriorating effects of cyclic loading on the soil properties shall be evaluated for the site-specific conditions and considered in the design where relevant. See also [10.1.5].

### 10.1.5 Soil-structure interaction

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

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

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

10.1.5.4 For dynamic analysis of the system of wind turbine, support structure and foundation, realistic stiffness values for the soil support of the foundation structure shall be applied. For example – in the case of pile foundations – p-y curves representative of the true physics of the pile-soil interaction, including realistic initial p-y stiffness, shall be applied. These requirements to realistic representation of stiffness also apply to assessment of the natural frequency of the system of wind turbine, support structure and foundation.

**Guidance note:**

The requirement for use of realistic 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, realistic stiffness value estimates can be obtained on the basis of back analyses of data for similar foundations installed in similar soils.

---end---of---Guidance---note---

### 10.2 Stability of seabed

#### 10.2.1 Slope stability

10.2.1.1 The risk of slope failure shall be evaluated. Such evaluations shall cover:

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

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

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

10.2.1.3 The safety against slope failure for ULS design shall be analysed using material factors ($\chi_M$):

$$\chi_M = \begin{cases} 1.15 & \text{for effective stress analysis} \\ 1.25 & \text{for total stress analysis} \end{cases}$$

#### 10.2.2 Hydraulic stability

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

10.2.2.2 An investigation of hydraulic stability shall assess the risk for:
— softening of the soil and consequent reduction of bearing capacity due to hydraulic gradients and seepage forces
— formation of piping channels with accompanying internal erosion in the soil
— surface erosion in local areas under the foundation due to hydraulic pressure variations resulting from environmental loads.

10.2.2.3 When erosion is likely to reduce the effective foundation area, measures shall be taken to prevent, control and/or monitor such erosion, as relevant, see [10.2.3].

10.2.3 Scour and scour prevention

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

Guidance note:
When a structure is placed on the seabed, the water-particle flow associated with steady currents and passing waves will undergo substantial changes. The local change in the flow will generally cause an increase in the shear stress on the seabed, and the sediment transport capacity of the flow will increase. In the case of an erodible seabed, this may result in a local scour around the structure. Such scour will be a threat to the stability of the structure and its foundation.

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

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

10.2.3.3 In an analysis of scour, the effect of steady current, waves, or current and waves in combination shall be taken into account as relevant.

Guidance note:
The extent of a scour hole will depend on the dimensions of the structure and on the soil properties. In cases where a scour protection is in place, it will also depend on the design of the scour protection.

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

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

Guidance note:
When scour protection consists of an earth structure, such as a sequence of artificially laid-out soil layers, it must be ensured that standard filter criteria are met when the particle sizes of the individual layers of such an earth structure are selected.

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

10.2.3.5 In cases where a scour protection is in place at a foundation structure and consists of an earth structure, the effect of soil support from the scour protection can be taken into account in the design of the foundation structure. For this purpose, a scour hole in the scour protection material shall be assumed with dimensions equal to those that are assumed in the design of the scour protection for the relevant governing ULS event.

10.2.3.6 A methodology for prediction of scour around a vertical pile that penetrates the seabed is given in App.J.

10.3 Pile foundations

10.3.1 General

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

10.3.1.2 In evaluation of soil resistance against pile loads, the following factors shall be amongst those to be considered:
— shear strength characteristics
— deformation properties and in-situ stress conditions of the foundation soil
— method of installation
— geometry and dimensions of pile
— type of loads.

10.3.1.3 The data bases of existing methods for calculation of soil resistance to axial and lateral pile loads are often not covering all conditions of relevance for offshore piles. This 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.

10.3.1.4 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, back analyses from similar piles in similar soil conditions.

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10.3.1.5 Structures with piled foundations shall be assessed with respect to stability for both operation and temporary design conditions, e.g. prior to and during installation of the piles.

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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10.3.1.6 Unless otherwise specified, the following material factors $\gamma_M$ 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>
<thead>
<tr>
<th>Type of geotechnical analysis</th>
<th>Limit state</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ULS</td>
<td>SLS</td>
</tr>
<tr>
<td>Effective stress analysis</td>
<td>$\gamma_M$</td>
<td>$\gamma_M$</td>
</tr>
<tr>
<td></td>
<td>1.15</td>
<td>1.0</td>
</tr>
<tr>
<td>Total stress analysis</td>
<td>1.25</td>
<td>1.0</td>
</tr>
</tbody>
</table>

10.3.1.7 For determination of design pile resistance against axial pile loads in ULS design, a material factor $\gamma_M = 1.25$ shall be applied to all characteristic values of pile resistance, i.e. to characteristic limit skin friction and characteristic tip resistance.

Guidance note:
This material factor may be applied to pile foundations of multi-legged jacket or template structures. The design pile loads shall 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 soil strength.

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

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 shall not include more piles that those supporting one specific leg.

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10.3.1.8 For drilled piles, the assumptions made for the limit skin friction in design shall be verified during the installation.

Guidance note:
The drilling mud 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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10.3.1.9 Laterally loaded piles may be analysed on the basis of realistic 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.

10.3.1.10 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 recognised plastic theorems so as to avoid nonconservative 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.

10.3.1.11 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 [10.3.1.6] shall be used.

10.3.1.12 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.

10.3.2 Design criteria for monopile foundations

10.3.2.1 For geotechnical design of monopile foundations, both the ultimate limit state and the serviceability limit state shall be considered.

10.3.2.2 For design in the ultimate limit state, design soil strength values are to be used for the soil strength, defined as the characteristic soil strength values divided by the specified materials factor. Design loads are to be used for the loads, each design load being defined as the characteristic load multiplied by the relevant specified load factor. The loads are to be representative of the extreme load conditions. Two cases are to be considered:

- axial loading
- combined lateral loading and moment loading.

10.3.2.3 For axial loading in the ULS, sufficient axial pile capacity shall be ensured.

**Guidance note:**

The pile head is defined to be the position along the pile in level with the seabed. Sufficient axial pile capacity can be ensured by checking that the design axial load on the pile head does not exceed the design axial resistance, obtained as the design unit skin friction, integrated over the pile surface area, plus a possible pile tip resistance.

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 ISO 19902 and DNV Classification Notes No. 30.4.

The effects of cyclic loading on the axial pile resistance should be considered in design. The main objective is to determine the shear strength degradation, i.e. the degradation of the unit skin friction, along the pile shaft for the appropriate prevailing loading intensities.

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.

---end---of---Guidance---note---

10.3.2.4 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. 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 can be ensured by means of a so-called single pile analysis in which the pile is 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. 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) can 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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10.3.2.5 For design in the serviceability limit state, characteristic soil strength values are to be used for the soil strength. Characteristic loads are to be used for the loads. The loading shall be representative of 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 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.

10.3.2.6 For design in the serviceability limit state, 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 seabed. 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 which is a requirement to the maximum allowable rotation of the pile head at the completion of the installation of the monopile.

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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10.3.2.7 The effect of permanent buckling and plastic hinges in monopile foundations needs to be analysed if relevant. Otherwise, permanent deformations in the monopile structure are not allowed.

10.3.3 Design criteria for jacket pile foundations

10.3.3.1 Jacket piles are the piles that support a jacket or frame structure such as a tripod platform. For geotechnical design of jacket piles, both the ultimate limit state and the serviceability limit state shall be considered.

10.3.3.2 For design in the ultimate limit state, design soil strength values are to be used for the soil strength, defined as the characteristic soil strength values divided by the specified materials factor. Design loads are to be used for the loads, each design load being defined as the characteristic load multiplied by the relevant specified load factor. The loads are to be representative of the extreme load conditions. Two cases are to be considered:

— axial loading
— combined lateral loading and moment loading

10.3.3.3 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.

**Guidance note:**

The verification of sufficient axial and lateral capacities of the individual piles can 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.

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The p-y curves can be generated according to procedures given in App.F for cyclic loading conditions. p-y curves established according to these procedures will automatically account for cyclic degradation effects in the lateral resistances.

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 App.F.

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.

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10.3.3.4 Pile group effects shall be accounted for.

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 soils leads to displacements of the soils 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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10.3.3.5 For design in the serviceability limit state, characteristic soil strength values are to be used for the soil strength. Characteristic loads are to be used for the loads. The loading shall be representative of 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. 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.

10.3.3.6 For design in the serviceability limit state, it shall be ensured that deformation tolerances are not exceeded.

Guidance note:
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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10.3.4 Design of piles subject to scour

10.3.4.1 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.

Guidance note:
The p-y and t-z curves must 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 are to be generated on the basis of a modified seabed level which is to 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 are to be applied.

Unless data indicate otherwise, the depth of a current-induced scour hole around a pile in sand can be assumed equal to a factor 1.3 times the pile diameter. For large-diameter piles such as monopiles, this emphasises the need for scour protection unless the piles are designed with additional lengths to counteract the effects of the scour.

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10.4 Gravity base foundations

10.4.1 General

10.4.1.1 Failure modes within the categories of limit states ULS and ALS shall be considered as described in [10.4.2].

10.4.1.2 Failure modes within the SLS, i.e. settlements and displacements, shall be considered as described in [10.4.3] using material coefficient $\gamma_M = 1.0$.

10.4.2 Stability of foundations

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

Guidance note:
For gravity base structures equipped with skirts which penetrate the seabed, the theoretical foundation base shall be assumed to be at the skirt tip level. Bucket foundations, for which penetrating skirts are part of the foundation solution, and for which suction is applied to facilitate the installation, shall be considered as gravity base structures for the condition after the installation is completed.

---end---of---Guidance---note---

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

10.4.2.3 For design within the applicable limit state categories ULS and ALS, the foundation stability shall be evaluated by one of the following methods:
— effective stress stability analysis
— total stress stability analysis.

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

10.4.2.5 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.

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

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

Guidance note:
Gravity base 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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10.4.2.8 For structures where skirts, dowels or similar foundation members transfer loads to the foundation soil, the contributions of these members to the bearing capacity and lateral resistance may be accounted for as relevant. The feasibility of penetrating the skirts shall be adequately documented.

10.4.2.9 Foundation stability shall be analysed in the ULS by application of the following material factors to the characteristic soil shear strength parameters:

\[ \kappa_M = 1.15 \text{ for effective stress analysis} \]
\[ \kappa_M = 1.25 \text{ for total stress analysis.} \]

10.4.2.10 Effects of cyclic loading shall be included by applying load factors in accordance with [10.1.1.6].

10.4.2.11 In an effective stress analysis, evaluation of pore pressures shall include:
— initial pore pressure
— build-up of pore pressures due to cyclic load history
— transient pore pressures through each load cycle
— effects of dissipation.
10.4.2.12 The safety against overturning shall be investigated in the ULS and in the ALS.

10.4.3 Settlements and displacements

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

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

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

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

10.4.4 Soil reactions on foundation structure

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

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

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

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

10.4.5 Soil modelling for dynamic analysis

10.4.5.1 Dynamic analyses of a gravity structure shall consider the effects of soil-structure interaction. For homogeneous soil conditions, modelling of the foundation soils 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 dynamic 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 App.G.

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

10.4.6 Filling of voids

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

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

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

— chemical
— mechanical
— placement problems such as incomplete mixing and dilution.
SECTION 11 CORROSION PROTECTION

11.1 Introduction

11.1.1 General

11.1.1.1 In this section, the requirements and guidance for corrosion control of offshore wind turbine structures are given.

11.1.1.2 Methods for corrosion control include corrosion allowance, cathodic protection, corrosion protective coatings and use of corrosion resistant materials. In closed internal compartments, corrosion may also be mitigated by control of humidity or depletion of oxygen. The term corrosion control further includes the inspection and maintenance of corrosion protection systems during operation.

**Guidance note:**
Corrosion control by exclusion of oxygen is primarily an option for structural compartments which are only externally exposed to seawater, e.g. internals of legs and bracings of jacket structures that are completely free-flooded at installation. Any compartments potentially exposed to air will need to be kept permanently sealed by welding or by maintenance of overpressure by nitrogen to prevent any air ingress.

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11.1.1.3 When corrosion allowance is part of the required corrosion protection, the corrosion allowance shall be considered in structural design for all limit state analyses by appropriate reduction of nominal thicknesses, see Sec.7.

11.2 Corrosion zones and applicable methods for corrosion control

11.2.1 Atmospheric zone

11.2.1.1 External and internal surfaces of steel structures exposed in the atmospheric zone, which is the zone extending above the splash zone as defined in [11.2.2], shall be protected by coating.

11.2.1.2 For internal surfaces of the atmospheric zone without control of humidity, a corrosion allowance as specified in [11.3.1] may replace coating. For any internal surfaces of structural parts defined as ‘primary’, the corrosion allowance shall then be based on a corrosion rate of minimum 0.10 mm/yr, unless special conditions justify a lower rate.

11.2.1.3 Corrosion resistant materials are applicable for certain critical components, for example stainless steel for bolting and other fastening devices, and GRP for grating.

11.2.2 Splash zone

11.2.2.1 The splash zone is the part of a support structure which is intermittently exposed to seawater due to the action of tide or waves or both. As a consequence of this action, the corrosive environment is severe, maintenance of corrosion protection is not practical and cathodic protection is not effective for parts of this zone. Special requirements for fatigue design of structural components exposed to the splash zone apply, see Sec.7.

11.2.2.2 The upper limit of the splash zone is the high still water level with a recurrence period of 1 year increased by the crest height of a reference wave whose height is equal to the significant wave height with a return period of 1 year. The lower limit of the splash zone is the low still water level with a recurrence period of 1 year reduced by the trough depth of a reference wave whose height is equal to the significant wave height with a return period of 1 year.

**Guidance note:**
The definition of the splash zone in [11.2.2.2] is in accordance with the definition given in IEC61400-3. The crest height and the trough depth of the reference wave used in the definition of the splash zone are location-specific and depend on the applied wave theory. The definition of the splash zone deviates from the definition used in EN 12495. The definition of the splash zone also deviates from the definition used in other DNV offshore codes, such as DNV-OS-C101. This alternative definition of the splash zone is included below for information only:
For specification of the upper and lower limits of the splash zone according to this definition, a reference wave height is specified as one-third of the 100-year wave height.

The upper limit of the splash zone $SZ_U$ is calculated as

$$SZ_U = U_1 + U_2 + U_3$$

in which

$U_1 = 60\%$ of the reference wave height
11.2.2.3 External and internal surfaces of steel structures in the splash zone shall be protected by a corrosion control system. The corrosion control system shall be suitable for resisting the aggressive environment in the splash zone which in certain areas may include drifting ice. Use of coating is mandatory for external surfaces of primary structures. Coating systems to be applied in the splash zone shall be made from manufacturer specific materials that have been qualified for the actual coating system by proven experience or relevant testing, e.g. according to NORSOK M-501. Maintenance of coating systems in the splash zone is not practical and coating of primary structures shall therefore be combined with a corrosion allowance. The corrosion allowance (CA) shall be calculated as

\[ CA = V_{corr} \cdot (T_D - T_C) \]

where \( V_{corr} \) is the expected maximum corrosion rate, \( T_C \) is the design useful life of the coating and \( T_D \) is the design life of the structure. To properly reflect the actual exposure time, \( T_D \) shall include the time between the installation of the structure and the installation of the wind turbine, typically 1 to 2 years, as well as the design life of the wind turbine, typically 20 years.

For internal surfaces of primary structures, use of coating is optional. The necessary corrosion allowance for internal surfaces shall be calculated according to the above formula, assuming \( T_C = 0 \) when no coating is used.

For parts of the splash zone located below MWL, cathodic protection may be assumed for design purposes to be fully protective and no corrosion allowance is required. Coatings for corrosion control in the splash zone shall as a minimum extend to MWL–1.0 m, taking into account the uncertainties in controlling this level during installation.

11.2.2.4 For primary structural parts to be operated in a temperate climate (annual mean surface temperature of seawater is 12°C or lower), the design corrosion rate of external surfaces in the splash zone above MWL shall be minimum 0.30 mm/yr, unless practical experience or other considerations indicates otherwise. In subtropical and tropical climates a design corrosion rate of minimum 0.40 mm/yr shall apply for such surfaces, unless practical experience indicates otherwise. For internal surfaces of primary structural parts in the splash zone, design corrosion rates shall be minimum 0.15 mm/yr in temperate climates and minimum 0.20 mm/yr in subtropical and tropical climates, unless practical experience or other considerations indicates otherwise.

11.2.2.5 For coating systems based on epoxy and meeting the requirements for coating materials and quality control of surface preparation and coating application in NORSOK M-501 Coating System No. 7 (min. DFT 350 mm), a useful life of up to 15 years may be assumed in the splash zone. For an equivalent system based on glass-flake reinforced epoxy or polyester (min. DFT 350 mm), a useful life of up to 20 years may be assumed in the splash zone.

11.2.2.6 For secondary structural parts in the splash zone, the needs for corrosion allowance may be assessed on an individual basis. The needs for corrosion allowance for such parts may depend on risk to human life, economical risk, possibility for inspection, maintenance and repair, and possibility for replacement. Provisions for replacement of corroded components may be considered as an alternative to a corrosion control designed for the service life of the support structure.

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11.2.2.7 For boat landings and access ladders, coating shall be applied in order to minimise nuisance, caused by corrosion, for service personnel. Combination with a corrosion allowance is recommended. As an alternative, corrosion resistant alloys may be considered for certain applications.

11.2.2.8 Bolts and other critical components in the splash zone for which corrosion allowance is not adequate shall be manufactured from corrosion resistant materials, see [11.5.2].

11.2.2.9 External surfaces in the splash zone below MWL shall have cathodic protection (CP).

11.2.3 Submerged zone

11.2.3.1 The submerged zone consists of the region below the lower limit of the splash zone, including the scour zone and the zone of permanently buried structural parts.

11.2.3.2 External surfaces of the submerged zone shall have cathodic protection (CP) in accordance with [11.4]. Use of coating is optional and is then primarily intended to reduce the required CP capacity. Manufacturer specific materials to be used for a coating system shall have documented compatibility with CP. The design of CP shall take into account possible scouring causing free exposure to seawater of surfaces initially buried in sediments. The design of CP shall also take into account current drain to all external surfaces to be buried in sediments. Internal surfaces of skirts and piles are not required to be included in current drain calculations for the external CP system. Steel surfaces buried in deep sediments need no corrosion protection, but will still drain current from a CP system due to the electrochemical reduction of water to hydrogen molecules on such surfaces. In the uppermost buried zone (about 0.1 m) anaerobic bacteria may cause corrosion which will be prevented by external CP.

11.2.3.3 Internal surfaces of the submerged zone shall be protected by either CP or corrosion allowance, with or without coating in combination. When CP is to be used, the CP shall be based on GACP according to [11.4.2], it shall be provided by anodes installed internally and it shall take into account current drain to any buried internal surfaces, such as internal surfaces of skirts, piles and J-tubes. Any corrosion allowance for primary structural parts with more or less free replenishment of seawater, or of air above the seawater surface, shall be determined based on a corrosion rate of minimum 0.10 mm/yr, unless practical experience or other considerations indicates otherwise. In the upper sediment zone, bacteria may cause a mean corrosion rate in excess of 0.10 mm/yr, and the application of a coating should be considered for this zone. With coating of most seawater-exposed internals, CP is applicable as a backup. In the internal splash zone, coating to be provided in combination with corrosion allowance may be accounted for as described in [11.2.2.3]. The requirement for a corrosion allowance on internal surfaces when no CP is used does not apply for permanently buried structural parts located deeper than 1 m below the seabed.

Guidance note:
Lower corrosion rates than 0.10 mm/yr may, in principle, apply in the atmospheric seawater-exposed zones of closed compartments than in the corresponding zones of open compartments. However, experience shows that, in practice, it is difficult to obtain compartments that will be completely sealed and airtight. Some compartments such as the interiors of monopiles are periodically accessed for inspection and repair and can therefore not be considered completely sealed. Also, large differences in tide may result in variations of the internal water level. In addition, even in virtual absence of oxygen in the seawater, corrosion by anaerobic bacteria can occur. It is recommended that these issues be taken into consideration when evaluating options for corrosion control for internal compartments.

11.3 Corrosion allowance

11.3.1 General

11.3.1.1 The extension of the corrosion zones defined in [11.2] and any corrosion allowances to apply in these zones for a specific project shall be specified in a dedicated section of a relevant project document, preferably the design basis. The specification of the corrosion zones shall take into account any uncertainties involved during the installation. The specification of corrosion allowance shall take into account the criticality of the structural part in question.

11.3.1.2 For surfaces of primary structural parts exposed in the splash zone and for internal surfaces of the submerged zone without CP, the corrosion allowance for surfaces with and without coating shall be calculated as specified in [11.2]. Any specified corrosion allowance shall be minimum 1.0 mm.

Guidance note:
Corrosion allowance applied for primary structural parts will affect any fatigue calculations to be performed, because the corrosion rate used to specify the corrosion allowance must comply with the assumed corrosion conditions which govern the S-N curve to be used for the fatigue calculations. In particular, this implies that if substantial metal loss is expected, free corrosion conditions must in general be assumed, and the “free corrosion” S-N curve is then required.
For further details, see Sec.7.

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11.3.1.3 For secondary structural parts, the needs for corrosion allowance may be assessed on an individual basis, see [11.2.2.7].

11.4 Cathodic protection

11.4.1 General

11.4.1.1 Cathodic protection of offshore structures by galvanic anodes (GACP) is well established and is generally preferred for such structures. Use of Impressed Current Cathodic Protection (ICCP) for such structures may offer certain advantages but is largely unproven for offshore wind turbine structures and there is no design standard available giving detailed requirements and advice as for galvanic anode systems. Even with adequate design, ICCP systems are more liable to environmental damage and third-party damage than GACP systems. Requirements for documentation of GACP and ICCP design are given in [11.4.2.5] and [11.4.3.7], respectively.

Guidance note:
Use of ICCP in lieu of GACP should be evaluated in a conceptual CP design report to be duly assessed by the owner prior to the implementation of ICCP for external corrosion control, see guidance note to [11.4.3].

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11.4.1.2 Use of GACP or ICCP and the applicable design standard for this use shall be specified in the project design basis.

11.4.2 Galvanic anode cathodic protection (GACP)

11.4.2.1 DNV-RP-B401 gives requirements and guidelines for cathodic protection design, anode manufacturing and installation of galvanic anodes. Alternative standards using the CP calculation procedure of DNV-RP-B401 may be used; however, the documentation of the design shall fully comply with the requirements given in [11.4.2.5], which contains amendments to the requirements given in DNV-RP-B401, Sec.7.13. In case DNV-RP-B401 has been specified for a GACP system, all design parameters affecting the CP current demand and the performance of anodes specified in DNV-RP-B401 shall apply, unless otherwise specified or accepted by the owner in writing. When DNV-RP-B401 is specified for CP design for structures located in waters with high seawater currents, such as in shallow waters with large differences between HAT and LAT, the initial design current densities for all initially bare steel surfaces should be considered for increase in order to account for the effect of high seawater currents. Initial design current densities are preferably to be specified in the design basis.

Guidance note:
The terminology for CP-related terms used in this section complies with that used in DNV-RP-B401.
In case reference is made to DNV-RP-B401 for CP design and no project-specific initial design current densities are specified in the design basis, the initial design current densities in Table 10-1 in DNV-RP-B401 are recommended to be increased by 50% for all initially bare steel surfaces in order to account for the effect of high seawater currents. The mean and final design current densities recommended in DNV-RP-B401 may be reduced to reflect provisions made for retrofitting of anodes as well as other factors reducing the need for inherent conservatism in the CP design.

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11.4.2.2 Cathodic protection by galvanic anodes shall utilise Al or Zn based materials with a composition in compliance with the applicable CP design standard or a specification issued or approved by the owner. Unless otherwise specified or accepted by the owner, the CP system shall have a design life which as a minimum shall be equal to the design life of the structure, including the period from installation to start of operation as well as the period of operation. Anodes to be used on a structure shall preferably be of identical or similar size (externally or internally). Reference is made to DNV-RP-B401, Item 7.8.6.

Guidance note:
The ratio between design current output and net anode mass for any anodes of different type or size should not differ by more than 5% unless early consumption of certain anodes is intentional.
In cases where the majority of the installed anodes in one support structure are clustered together rather than distributed evenly, special care must be exercised to make sure that the chemical compositions of the anodes are within a sufficiently narrow compositional limit thereby to ensure uniform current output and consumption.

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11.4.2.3 All anodes shall be located minimum 1.0 m below LAT and minimum 1.0 m above the seabed with due consideration of possible variation in the seabed level owing to moving sand dunes. For calculation of initial current demand in areas with large tidal zones, the surface area up to HAT should be considered for CP
design. The design of anode supports and their fastening to the structures shall take into account the applicable forces during installation and operation of the structure. For structural design, reference is made to Sec.7. A reference to structural design calculations for anode fastening shall be included in the CP design report. In case replacement of anodes is foreseen, provisions for subsea fastening of new anodes shall be described in the design documentation. The anodes shall be distributed to avoid interference reducing their current output in accordance with the applicable CP design standard. In case there are reasons to assume a significant interaction between anodes, an analysis by a computer model should be carried out to determine a reduction factor for the anode current output, see guidance note to [11.4.3.3]. The reduction factor is to be applied as a factor on anode current output as calculated by anode resistance formulas for anodes with such interaction.

**Guidance note:**

For largely uncoated monopiles in waters with depths larger than approximately 20 to 25 m, it may be difficult to achieve sufficient external cathodic protection at the seabed from anodes located solely at the transition piece between tower and pile due to a large number of anodes required and a limited surface area available for distribution of anodes. Complete or partial coating of the external seawater-exposed pile surface may then be applied to reduce the current demand for the initial exposure period. For this purpose, even “Category I” coating according to DNV-RP-B401 can be considered.

---end---of---Guidance---note---

11.4.2.4 After maximum 365 days, a CP survey shall be performed to confirm that the structures for which the GACP design was 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.

**Guidance note:**

For the CP survey of a few representative structures, it is recommended as a minimum to survey one structure for every 20 installed structures. However, the actual number of structures to be surveyed should reflect what in each case is deemed necessary in order to obtain the required representativeness of the survey. All potential sources for variation from structure to structure should be considered. One issue to consider is the relative positioning and distances between the structures in a wind farm together with differences in environmental conditions. Another issue is the duration of the installation period which in large wind farms may span more than one year.

For the CP survey, it is sufficient to establish an adequate global protection level extending from the uppermost part of the submerged zone to the seabed. Positioning of a reference electrode by means of a diver or an ROV is then not strictly necessary, provided the weather and sea current conditions allow the positioning of the reference electrode within a few metres of the steel surface. On the other hand, if marginal protection is indicated (protection potentials less negative than \(-0.90 \text{ V rel Ag/AgCl/seawater}\)), the survey should be extended to include close potential recordings, less than 0.5 m from the steel surface, and focus on locations as far as possible from the anodes, including locations at the seabed. For steady-state conditions, which may require more than 180 days for largely uncoated structures, a protection potential more negative than \(-0.90 \text{ V (corrected for any IR drop)}\) is an indication of adequate functioning of a GACP system. There is no documentation that a potential (IR free) in the range \(-0.80 \text{ to } -0.90 \text{ V}\) has ever led to any corrosion damage (including corrosion damage by bacteria) but a potential in this range will cause enhanced current output and consumption of anodes. The enhanced current output and consumption of anodes may not match the mean current density which is used for design and which is lowest in the potential range \(-0.90 \text{ to } -1.00 \text{ V}\) (as aimed for in DNV-RP-B401).

Recordings of anode potential in the immediate vicinity of the anode surface should also be included in a close potential survey. For CP monitoring of monopiles with anodes located on the transition piece and with the reference electrode connected to the transition piece, corrections should be made for any significant voltage (IR) drop through continuity cables from the transition piece to the pile.

The detailed planning and execution of initial potential survey and the evaluation of the results will not require the involvement of the designer of the GACP system.

---end---of---Guidance---note---

11.4.2.5 The GACP design shall be documented in a dedicated report containing the following items:

— Design premises (with reference to the project design basis and other relevant project specifications, codes and standards), specified or approved by the owner, including design life and any modifications of CP design parameters specified or recommended in the applicable CP design standard
— Calculations of surface areas and current demands (initial/final and mean), including current drain to sediment buried surfaces
— Calculations of anode resistance and current output (initial/final) of the anode(s) to be used
— Assessment of any anode interactions for external CP according to [11.4.2.3] and modelling of protection potential distribution for any internal CP
— Tentative anode drawing(s) and drawing(s) showing dimensions and locations of anode cores and any provisions for electrical continuity by other means than welding (if applicable)
— Calculations of required net anode mass and anode current output to meet the calculated current demands, and the required number of anodes to meet both requirements.
— Calculations and assessment of voltage drop across electrical continuity cables (if applicable)
— Drawings showing distribution of anodes and locations of any electrical continuity cables
— Requirements for manufacturing of anodes (e.g. by reference to a standard or a project specification)
— Requirements for installation of anodes (e.g. by reference to a standard and project specific procedures) and assessment of structural integrity, see [11.4.2.3]
— Recommendations, requirements and draft method statement for the initial potential survey, including reporting requirements.

Calculation spread sheets for CP design may be contained in an appendix to the report; however, the design parameters and the results of all calculations referred to above shall be compiled in the report.

11.4.3 Impressed current cathodic protection (ICCP)

11.4.3.1 In addition to adequate CP potential and current distribution, the detailed design of an Impressed Current CP (ICCP) system shall focus on the long term mechanical integrity of the equipment, including impressed current anodes, reference electrodes, cables and connectors, with due consideration of environmental parameters, primarily wave forces and sea currents.

**Guidance note:**
The detailed design of an ICCP system should be preceded by a conceptual design activity for the owner to conclude that an ICCP system is preferred for the specific project and is to be included in the project design basis. This can be done by taking costs for installation and operation into account, and by including other relevant considerations. Considerations for the selection of GACP or ICCP may include effects of weight and drag forces of GACP systems, availability of a continuous current source for ICCP, mechanical integrity of anodes, reference electrodes and cables of ICCP systems, and environmental effects (release of Zn from galvanic anodes and of active chlorine from impressed current anodes). Guidance for the design of ICCP systems for offshore structures is given in NACE SP0176 and EN 12495.

11.4.3.2 Unless otherwise specified or agreed by owner, ICCP anodes and reference electrodes shall be designed to be replaceable if damaged or degraded by environmental effects or by other effects. In any case, long-term reliability of anodes, reference electrodes, subsea electrical couplings and cables shall be documented by the designer of the ICCP system making reference to a standard or documented performance of manufacturer specific equipment. The ICCP design shall duly consider needs for contingency due to damage or malfunction of individual anodes or reference electrodes. Based on design current demands (initial and final) applicable to GACP systems, a contingency corresponding to a minimum of 150% anode current capacity shall be included. A minimum of two reference electrodes shall be provided to control current output of each rectifier.

11.4.3.3 The ICCP design shall demonstrate that the capacity of the current source(s) and the anodes are adequate to achieve and maintain cathodic protection of all submerged parts of the structure, including current drain to sediment buried areas. The design shall aim for adequate protection to be effected within 30 days from commissioning of the ICCP system. This shall be achieved without exposing steel surfaces to more negative potentials than –1.10 V rel. Ag/AgCl/seawater which may otherwise lead to damage of any paint coating and possibly also to hydrogen induced damage to the steel structure. Adequate potential distribution shall be confirmed by computer based modelling of cathodic protection and utilising some empirical time-dependent relation between the cathodic current density and the protection potential. The CP modelling shall further demonstrate that the number and location of fixed reference electrodes is adequate to confirm that the structure is protected as required by the design.

**Guidance note:**
There are no standards defining the time dependent relation between potential and cathodic current density; hence, such relations are chosen at the discretion of the provider of the CP modelling tool. The provider of CP modelling tools should validate the reliability of the tools, e.g. by a potential survey of a real structure that has been subject to such modelling.

11.4.3.4 Dielectric shields are used to avoid overprotection close to ICCP anodes and to facilitate adequate current distribution. In the immediate vicinity of anodes, a prefabricated polymeric sheet is normally applied whilst a relatively thick layer of a special paint coating is applied as an outer shield, extending to the range of overprotection, i.e. protection potentials more negative than –1.15 V rel. Ag/AgCl/seawater according to the computer CP modelling. The selection of the particular coating shall then be justified by long term testing of resistance to cathodic disbondment at the most negative potential that (according to the CP modelling) may apply at the edge of the innermost shield of polymeric sheeting.

**Guidance note:**
Whilst the innermost dielectric shield (homogeneous polymeric sheet or polymer-lined steel sheet) is supplied by ICCP anode manufacturer, the outer shield (special paint coating) is normally applied by the fabrication contractor for the support structure. Quality control of surface preparation and coating application is then essential and should preferably be supervised by the manufacturer of the coating material or a third-party specialist.

---end-of-Guidance-note---
11.4.3.5 ICCP systems shall be designed for remote control of anode current output based on recordings from fixed reference electrodes. In addition to potential data, rectifier current output and voltage shall be recorded remotely, e.g. with a frequency of one or a few hours, and stored for easy retrieval and display. The rectified current output may be set to a fixed value based on an evaluation of recordings from fixed reference electrodes (minimum two per rectifier). As an option, the rectifier current output may be controlled automatically based on recordings from one or more reference electrodes. Minimum two reference electrodes per rectifier shall be provided for both options and with alarm functions for minimum/maximum potentials recorded by the reference electrodes.

11.4.3.6 ICCP systems shall be commissioned according to a project specific procedure. After minimum 30 days and maximum 365 days, a detailed CP survey shall be performed to confirm that the structures for which the ICCP design was 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, but rectifier parameters shall be checked for all structures and a potential survey shall be carried out if any inconsistency is established.

Guidance note:
For the CP survey of a few representative structures, it is recommended as a minimum to survey one structure for every 20 installed structures. However, the actual number of structures to be surveyed 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 duration of the installation period which in large wind farms may span more than one year.

For the CP survey, the reference electrode will need to be positioned by an ROV at locations close to and remote to the ICCP anodes. (Use of divers is restricted by safety hazards when approaching ICCP anodes.) For a large number of similar units in the same area, it is only necessary to carry out a survey of a number of structures which are considered to be representative.

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11.4.3.7 The designer of the ICCP system shall be involved in the planning and execution of the initial potential survey and in the evaluation of the results.

Guidance note:
The involvement of the ICCP designer in the execution of the initial potential survey should be specified in the agreed scope of work for the design of the ICCP.

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11.4.3.8 The ICCP design shall be documented in a dedicated report containing the following items:

— Design premises (with reference to the project design basis and other relevant project specifications, codes and standards), specified or approved by the owner, including design life and any modifications of CP design parameters specified or recommended in the applicable CP design standard
— Calculations of surface areas and current demands (initial/final and mean), including current drain to sediment buried surfaces and with considerations for needs of contingency of current output capacity (due to e.g. damage/malfunction of one or more anodes)
— Calculations of anode current output based on rectifier voltage, voltage drops in cables and anode resistance
— Results from CP modelling with coloured graphs showing the development of the protection potential (e.g. after 10, 100 and 1000 hours) with emphasis on potentials at the edges of the inner and outer dielectric shields, at reference electrodes and at remote locations from the anodes)
— Justification of design life for anodes, fixed reference electrodes, cables, electrical connectors and other items with restricted access for maintenance and repairs
— Specification and drawings for rectifier unit or other current sources
— Specifications and detailed drawings of anodes, dielectric shields, fixed reference electrodes and outline of electric circuiting
— Material Data Sheets for cables, electrical connectors, dielectric shield materials, fastening devices and other equipment
— Drawings showing distribution of anodes, reference electrodes and routing of cables, including cable conduits
— Requirements for installation of anodes, reference electrodes and cables by welding to primary and secondary structural components (e.g. by reference to a standard and project specific procedures)
— Description of system for monitoring of protection potential and control of anode current output and logging of potential recordings and rectifier current and voltage
— Procedures for commissioning and recommendations (method statement) for the initial potential survey
— General description of operational procedures, including replacement of anodes and reference electrodes, logging of data and periodic inspection and maintenance of current sources.
11.5 Corrosion protective coatings and corrosion resistant materials

11.5.1 Corrosion protective coatings

11.5.1.1 Requirements for corrosion protective coatings shall be specified in a dedicated document or in a section of some other relevant design document. At least for primary structural parts, generic types of coating systems and requirements to the qualification of manufacturer specific materials to be used for such systems (see [11.5.1.2]) should be specified in the project design basis.

Guidance note:
Documentation of requirements for type of corrosion protective coating systems and for qualification of manufacturer specific coating materials is required in order to allow for subsequent verification of the overall corrosion control system. No explicit requirements for corrosion protective coating of individual parts are given in this standard, because requirements to the coating influence the need for corrosion allowance and thus in turn influence the design of the structure against fatigue. Without explicit requirements to coating, the designer is free to choose a lower cost coating system with less useful life provided that he compensates by applying a larger corrosion allowance.

---end of Guidance note---

11.5.1.2 For structural parts in each corrosion zone, the selection of coating systems as defined in, for example, NORSOK M-501 or ISO 12944 shall be specified, as well as requirements for the qualification of manufacturer specific coating materials and of personnel to carry out coating work. The specification shall further contain general requirements for the quality control of coating work and for the coating applicator’s documentation to be provided prior to, during and after completion of the work.

Guidance note:
A coating system is defined by generic types of coating materials, by thickness of individual layers and by surface preparation. The term “coating work” includes inspection and testing of surface preparation and coating application plus coating repairs.

---end of Guidance note---

11.5.1.3 Coating systems and manufacturer specific coating materials for the splash zone shall be selected with due consideration of the conditions that may apply for the specific project including any exposure to floating ice and procedures for periodic removal of marine growth.

11.5.1.4 Hot dip zinc coating is applicable to certain secondary structural parts in the atmospheric zone and in the splash zone. With a specified minimum thickness of 50 µm and compliance with ISO 1461, the useful life of zinc coating exposed externally may be assumed to be minimum 5 years in the splash zone and minimum 10 years in the external atmospheric zone. Any polymeric coatings to be used for fasteners exposed externally in the atmospheric and splash zones should have an inner layer of electrolytic zinc and defined requirements to surface preparation prior to coating.

11.5.2 Corrosion resistant materials
Any corrosion resistant materials to be used should be specified by reference to a materials standard (e.g. ASTM) defining requirements to chemical composition, mechanical properties and quality control of manufacturing.

Guidance note:
Stainless steels to be used in the atmospheric zone or the splash zone should have a corrosion resistance equivalent or better than that of type AISI 316. Bolts of this material exposed to sea spray but sheltered from direct rainfall have sometimes suffered corrosion attacks and higher alloyed materials, such as type 25Cr duplex, should be considered for critical applications.

---end of Guidance note---
SECTION 12 TRANSPORT AND INSTALLATION

12.1 Marine operations

12.1.1 General

12.1.1.1 This section gives requirements and guidelines for planning and execution of marine operations, so that these operations are planned and carried out within defined and recognised safety level and with as small risk of failure as practical.

12.1.1.2 Marine operations in this context are defined as non-routine operations of limited defined duration, carried out for overall handling of an object at sea (offshore, inshore and at shore). Marine operations are normally related to handling of objects during temporary phases from or to the quay side or construction sites to final destination or installation site. Marine operations include activities such as load transfer operations, transport, installation, decommissioning and deconstruction.

12.2 Risk management during marine operations

12.2.1 Risk management in transportation and installation phases

12.2.1.1 Marine operations, including all support activities, should be thoroughly assessed already at conceptual design stages. Marine operations and the handled objects should be designed with due consideration to resisting characteristic loads and conditions as well as being practicable and safe. The planning process should address redundancy and backup philosophies, as well as all other activities required to reach an acceptable risk level.

12.2.1.2 The planning process should include assessment of personnel exposure and possibilities for reducing this through use of remotely operated tools and handling systems.

12.2.1.3 The project risk should be managed through a planned risk management process. A Risk Management Plan (RMP) should be prepared in order to describe, communicate and document activities and processes necessary for managing through all project phases, the risk involved in the planned marine operations.

12.2.1.4 DNV-RP-H101 gives guidance on how to manage and mitigate risks related to marine operations.

12.3 Marine warranty surveys

12.3.1 Purpose and regulations

12.3.1.1 A Marine Warranty Survey (MWS) may be required by the insurance company in order to effect an insurance for temporary phases such as sea transport and installation.

12.3.1.2 The purpose of an MWS is to ensure that the marine operations are performed within defined risk levels. These risk levels should be tolerable to marine insurance and also to the industry, as well as to the national and international Regulatory Bodies.

12.3.1.3 An MWS should be carried out in accordance with an internationally recognised standard. The DNV ‘Rules for Planning and Execution of Marine Operations’ represents a standard which is widely recognised by both insurance, finance and marine industries.

12.3.1.4 DNV ‘Rules for Planning and Execution of Marine Operations’, Pt.1 Ch.1, describes in detail the principles, the scope and the procedures for marine warranty surveys. See also [12.4].

12.3.1.5 Project Certification of wind farms does not include MWS. However, there are some synergy effects when DNV delivers both services. When DNV performs the MWS, the DNV surveyor for Project Certification will normally be present only during the first load-out and installation. The remaining surveys will be covered by the Marine Warranty surveyor.

12.4 Marine operations – general requirements

12.4.1 General

This subsection presents the parts of DNV ‘Rules for Planning and Execution of Marine Operations’ which are relevant for the temporary phases for wind turbine structures.

12.4.2 Planning of operations

12.4.2.1 Marine operations should be planned and prepared to bring an object from one defined safe condition
to another according to safe and sound practice, and according to defined codes and standards. The planning of the operations should cover planning principles, documentation and risk evaluation. The planning and design sequence is given in Figure 12-1.

**Figure 12-1**
Planning and design sequence

12.4.2.2 Operational prerequisites such as design criteria, weather forecast, organisation, marine operation manuals as well as preparation and testing should be covered.

12.4.2.3 Requirements to planning of operations are given in ‘DNV Rules for Planning and Execution of Marine Operations’, Pt. 1 Ch. 2.

### 12.4.3 Documentation

12.4.3.1 Acceptable characteristics shall be documented for the handled object and all equipment, temporary or permanent structures, vessels etc. involved in the operation.

**Guidance note:**
Note that all elements of the marine operation shall be documented. This also includes onshore facilities such as quays, soil, pullers and foundations.

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12.4.3.2 Properties for object, equipment, structures, vessels etc. may be documented with recognised certificates. The basis for the certification shall then be clearly stated, i.e. acceptance standard, basic assumptions, dynamics considered etc., and shall comply with the philosophy and intentions of DNV ‘Rules for Planning and Execution of Marine Operations’.

12.4.3.3 Design analysis should typically consist of various levels with a “global” analysis at top level, and with strength calculations for details as a lowest level. Different types of analysis methods and tools may apply for different levels.

12.4.3.4 Operational aspects shall be documented in the form of procedure, operation manuals, certificates, calculations etc. Relevant qualifications of key personnel shall be documented.

12.4.3.5 All relevant documentation shall be available on site during execution of the operation.

12.4.3.6 The documentation shall demonstrate that philosophies, principles and requirements of DNV ‘Rules for Planning and Execution of Marine Operations’ are complied with.

12.4.3.7 Documentation for marine operations shall be self-contained or clearly refer to other relevant documents.

12.4.3.8 The quality and details of the documentation shall be such that it allows for independent reviews of plans, procedures and calculations for all parts of the operation.
Guidance note:
A document plan describing the document hierarchy and scope for each document is recommended for major marine operations.

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12.4.3.9 Applicable input documentation such as:
— statutory requirements
— rules
— company specifications
— standards and codes
— concept descriptions
— basic engineering results (drawings, calculations etc.)
— relevant contracts or parts of contracts

should be identified before any design work is performed.

12.4.3.10 Necessary documentation shall be prepared to prove acceptable quality of the intended marine operation. Typically, output documentation consists of:
— planning documents including design briefs and design basis, schedules, concept evaluations, general arrangement drawings and specifications
— design documentation including load analysis, global strength analysis, local design strength calculations, stability and ballast calculations and structural drawings
— operational procedure including testing program and procedure, operational plans and procedure, arrangement drawings, safety requirement and administrative procedures
— certificates, test reports, survey reports, NDE documentation, as built reports, etc.

12.4.3.11 Execution of marine operations shall be logged. Samples of planned recording forms shall be included in the marine operations manual.

12.4.3.12 Further requirements are given in DNV ‘Rules for Planning and Execution of Marine Operations’ Pt.1 Ch.2.

12.4.4 Vessel stability

12.4.4.1 Sufficient stability and reserve buoyancy shall be ensured for all floating objects in all stages of the marine operations. Both intact and damaged stability shall be documented. For operations where stability and/or buoyancy at some stage is critical, special consideration shall be given to the duration of the critical condition, the risk of possible hazards and to the mobilisation time for – and amount of – backup systems.

12.4.4.2 Requirements for stability are given in ‘DNV Rules for Planning and Execution of Marine Operations’ Pt.1 Ch.2.

12.4.5 Systems and equipment

12.4.5.1 Systems and equipment shall be designed, fabricated, installed and tested in accordance with relevant codes and standards. Systems and equipment shall be based on a thorough consideration of functional and operational requirements for the complete operation. Emphasis shall be placed on reliability and contingency.

12.4.5.2 Requirements for stability are given in ‘DNV Rules for Planning and Execution of Marine Operations’ Pt.1 Ch.2.

12.4.6 Design loads

12.4.6.1 The design loads are loads or load conditions which form the basis for design and design verification.

12.4.6.2 The design loads include basic environmental conditions like wind, wave, current and tide. The design process involving characteristic conditions, characteristic loads and design loads is illustrated in Figure 12-2.
12.4.6.3 The load analysis should take into account dynamic effects and non-linear effects. Permanent loads, live loads, deformation loads, environmental loads as well as accidental loads should be considered.

12.4.6.4 Further requirements are given in DNV ‘Rules for Planning and Execution of Marine Operations’ Pt.1 Ch.3.

12.4.7 Structural design

12.4.7.1 Prerequisites for structures involved in marine operations shall include design principles, strength criteria for limit state design, testing, material selection and fabrication.

12.4.7.2 Requirements and guidelines are given in DNV ‘Rules for Planning and Execution of Marine Operations’ Pt.1 Ch.4.

12.4.8 Other issues

12.4.8.1 The installation of support structures for wind turbines in a large wind farm may pose some logistic challenges and it is recommended to carry out a logistics study for the installation and to plan the logistics of the transportation and installation carefully.

Guidance note:
There might be a need for inshore storage premises for intermediate storage of structures and structural components awaiting a weather window for installation.

The seabed topography and water depth, near shore as well as along the transportation route, may pose requirements for how the transportation to site can be carried out.

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12.4.8.2 It shall be assessed which initiatives and actions can be taken in design in order to facilitate the installation of the structure.

Guidance note:
Optimal planning and execution of the logistics for installation of many structures in a wind farm is a key issue, e.g. with a view to avoid costly waiting times.

It may in some cases contribute to an efficient installation without costly delays if the structural design is carried out such that the same weather criterion can be used for installation of support structures as for wind turbines and cables. This can be achieved by suitable planning and design.

However, if some operations can be performed in higher waves than other operations, the implication may be a larger flexibility of the installation activities, provided none of the activities are subject to stringent restrictions.

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12.5 Marine operations – specific requirements

12.5.1 Load transfer operations

12.5.1.1 The load transfer operations cover load-out, float-out, lift-off and mating operations.

12.5.1.2 Requirements for load transfer operations are given in DNV-OS-H201.
12.5.2 Sea transports

12.5.2.1 Specific requirements and guidelines for single-vessel and barge-towing operations are given in DNV ‘Rules for Planning and Execution of Marine Operations’, Pt.2 Ch.2.

12.5.2.2 Requirements and recommendations for transport on-board ship, towing of multi-hull vessels, self-floating and self-propelled carrier transports are given in DNV ‘Rules for Planning and Execution of Marine Operations’, Pt.2 Ch.3.

12.5.3 Offshore installation

12.5.3.1 Specific requirements and recommendations for offshore installation operations applicable for wind turbines and their support structures and structural components are given in DNV-OS-H204.

12.5.3.2 Environmental loads and load cases to be considered are described as well as on-bottom stability requirements and requirements to structural strength.

12.5.3.3 Operational aspects for ballasting, for pile installation and for grouting shall be considered.

12.5.4 Lifting operations

12.5.4.1 Guidance and recommendations for lifting operations, onshore, inshore and offshore, of objects with weight exceeding 50 tonnes are given in DNV-OS-H205.

12.5.4.2 The chapter describes in detail the basic loads, dynamic loads, skew loads and load cases to be considered. Design of slings, grommets and shackles as well as design of the lifted object itself are covered. In addition, operational aspects such as clearances, monitoring of lift and cutting of sea fastening are described.

12.5.5 Subsea operations

Subsea operations are relevant for subsea components related to floating wind turbine installation and tie-in of, for example, electrical cables. Planning, design and operational aspects for such installations are described in DNV ‘Rules for Planning and Execution of Marine Operations’, Pt.2 Ch.6.
SECTION 13 IN-SERVICE INSPECTION, MAINTENANCE AND MONITORING

13.1 Introduction

13.1.1 General

13.1.1.1 An offshore wind farm is typically planned for a 20-year design lifetime. In order to sustain the harsh offshore environment, adequate inspection and maintenance have to be carried out. This applies to the entire wind farm including substation and submerged power cables.

13.1.1.2 This section provides requirements and recommendations for inspection and maintenance of wind turbines, support structures, and submerged power cables. Requirements and recommendations for inspection of substations are given in DNV-OS-J201.

Guidance note:
Substations in wind farms are vital structures, which are usually designed to a higher safety level than wind turbines and support structures for wind turbines, and which require a more thorough inspection regime than what is needed for the wind turbines and their support structures.

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13.1.1.3 A program for inspection of the wind turbines and their support structures in a wind farm shall be defined and implemented. Programs for in-service inspections of wind turbines and their 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.

13.1.1.4 For single wind turbine structures and for wind farms comprising only a few wind turbine structures, it may be feasible to define rigid inspection programs with requirements for annual inspections and other periodical surveys which cover all turbines and structures in the wind farm. For large numbers of wind turbine structures in large wind farms, such rigid inspection programs will 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.

13.2 Periodical inspections – general

13.2.1 General

13.2.1.1 The provisions of this subsection apply to wind turbines, support structures and cables for which periodical inspections are chosen as the approach to in-service inspection.

13.2.1.2 The following periodical inspections shall be performed in order to evaluate the condition of the offshore wind farm during its design lifetime:

— periodical inspection of wind turbines
— periodical inspection of structural systems above water
— periodical inspection of structures below water
— periodical inspection of submerged power cables.

The periodical inspection consists of three levels of inspection, viz. general visual inspection, close visual inspection and non-destructive examination. General visual underwater inspections can be carried out using an ROV (Remotely Operated Vehicle), whereas close visual underwater inspections require inspections carried out by a diver.

13.2.2 Preparation for periodical inspections

13.2.2.1 A 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 components, systems and installations in the offshore wind farm will be covered by annual inspections over the five-year period.

13.2.2.2 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 as specified in the Certificate of Compliance.

13.2.2.3 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 reported or dealt with.
Guidance note:
The inspection will typically consist of an onshore part and an offshore part.
The onshore 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 offshore program, based on findings from the onshore part and systems selected from the Long Term Inspection Program.
The offshore 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.

13.2.3 Interval between inspections
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.

13.2.4 Inspection results
The results of the periodical inspections shall be assessed and remedial actions taken, if necessary. Inspection results and possible remedial actions shall be documented.

13.2.5 Reporting
The inspection shall be reported. The inspection report shall give reference to the basis for the inspection such as 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.

13.3 Periodical inspection – detailed

13.3.1 General

13.3.1.1 The provisions of this subsection apply to wind turbines, support structures and cables for which periodical inspections are chosen as the approach to in-service inspection.

13.3.1.2 The manufacturer’s service manual for the wind turbines shall be consulted for its specification of requirements for inspections of the wind turbines and their support structures.

13.3.2 Interval between inspections of wind turbines
The interval between inspections above water should not exceed one year. Requirements in the wind turbine service manual shall be followed.

13.3.3 Scope for inspection of wind turbines

13.3.3.1 The following items shall be covered by the inspection:
- blades
- gear boxes
- lifting appliances
- fatigue cracks
- dents and deformation(s)
- bolt pre-tension
- status on outstanding issues from previous periodical inspections of wind turbines.

13.3.3.2 Inspections as required in the wind turbine service manual come in addition to the inspection implied by [13.3.3.1].

13.3.4 Interval between inspections of structural systems above water
The interval between inspections above water should not exceed one year. Requirements in the wind turbine service manual shall be followed.
13.3.5 Scope for inspection of structural systems above water

13.3.5.1 The following items shall be covered by the inspection:

— tower structures
— transition pieces
— grouted connections
— lifting appliances
— access platforms
— upper part of J-tubes
— upper part of ladders
— upper part of fenders
— heli-hoist platforms
— corrosion protection systems
— marine growth
— fatigue cracks
— dents
— deformation(s)
— bolt pre-tension
— status on outstanding issues from previous periodical inspections above water.

13.3.5.2 Inspection for fatigue cracks at least every year as required by the lists in [13.3.3.1] and [13.3.5.1] 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 Sec. 7 Table 7-15, 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 design fatigue factor DFF as follows:

\[ \text{Inspection interval} = \frac{\text{Calculated fatigue life} \times \text{DFF}}{3.0}. \]

This implies the following requirements to inspection:

- \( \text{DFF} = 3.0 \): No check for fatigue cracks is needed, corresponding to an assumption of no access to the structural detail.
- \( \text{DFF} = 2.0 \): 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 \( \text{DFF} = 3.0 \) without inspections.
- \( \text{DFF} = 1.0 \): 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 \( \text{DFF} = 3.0 \) without inspections.

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13.3.5.3 Guidance for inspection of grouted connections is given in [13.4.1.4].

13.3.5.4 Inspections as required in the wind turbine service manual come in addition to the inspection implied by [13.3.2].

13.3.6 Interval between inspections of structures below water

The interval between inspections below water should not exceed five years.

Guidance note:
Five-year inspection intervals are common; however, more frequent inspections during the first few years after installation are recommended.

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13.3.7 Scope for inspection of structures below water

13.3.7.1 The following items shall be covered by the inspection:

— support structures
— lower part of J-tubes
— lower part of ladders
— lower part of fenders
— corrosion protection systems (anodes, coating etc.)
— marine growth
— fatigue cracks
— scour and scour protection
— damages and dents
— deformations
— debris
— status on outstanding issues from previous periodical inspections below water.

Visual inspections may be carried out by a remotely operated vehicle (ROV).

13.3.7.2 Inspection for fatigue cracks at least every five years as required by the list in [13.3.1.1] 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 Sec.7 Table 7-15, 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 factor $\gamma_m$ as follows:

$$\text{Inspection interval} = \frac{\text{Calculated fatigue life} \cdot \gamma_m^5}{1.25^5}.$$ 

This implies the following requirements to inspection:

- $\gamma_m = 1.25$ No check for fatigue cracks is needed, corresponding to an assumption of no access to the structural detail.
- $\gamma_m = 1.15$ 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 $\gamma_m = 1.25$ without inspections.
- $\gamma_m = 1.0$ 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 $\gamma_m = 1.25$ without inspections.

13.3.7.3 The protection potential shall be measured and fulfil minimum requirements.

13.3.7.4 If deemed critical, steel wall thickness shall be measured.

13.3.8 Interval between inspections of submerged power cables

The interval between inspections of submerged power cables should not exceed five years. A risk assessment can be carried out for determination of the appropriate inspection interval, such that export cables will be inspected frequently and array cables less frequently. Further, subsea cables buried in very stable seabed may not need as much monitoring as subsea cables buried in less stable sediments or in areas with significant tide.

13.3.9 Scope for inspection of submerged power cables

13.3.9.1 Array cables between the wind turbines and the transformer station as well as export cables to the shore shall be inspected, unless they are buried.

13.3.9.2 To the extent that submerged power cables are to be buried, it shall be ensured that the cables are buried to design depth.

13.4 Inspections according to risk-based inspection plan

13.4.1 General

13.4.1.1 The provisions of this subsection apply to wind turbines and support structures for which inspections of a few representative turbines and structures in a large wind farm and according to a risk-based inspection plan are chosen as the approach to in-service inspection.

13.4.1.2 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.
13.4.1.3 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 all structures in the wind farm shall be carried out.

13.4.1.4 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.

13.5 Deviations

13.5.1 General

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  STRESS CONCENTRATION FACTORS FOR TUBULAR JOINTS

Reference is made to DNV-RP-C203.
APPENDIX B  LOCAL JOINT FLEXIBILITIES FOR TUBULAR JOINTS

B.1 Calculation of local joint flexibilities

B.1.1 General

B.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 B-1. LJFs influence the global static and dynamic structural response.

B.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 B-1 provides information of degrees of freedom for which cross terms of local joint flexibility exist between different brace ends.

B.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.

B.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:

\[
LJF_{axial} = \frac{f_{axial}}{E D} \\
LJF_{IPB} = \frac{f_{IPB}}{E D} \\
LJF_{OPB} = \frac{f_{OPB}}{E D^3}
\]

in which \( E \) denotes Young’s modulus of elasticity, \( D \) 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.

B.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.

B.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

\[ \text{LJF} = \lambda_Y \text{LJF}_Y + \lambda_X \text{LJF}_X + \lambda_K \text{LJF}_K \]

in which the \( \lambda \) values are the fractions corresponding to the joint type designated by the subscript when the joint is classified by loads.

B.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.

B.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.

B.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.

B.1.1.10 The parametric expressions for calculation of LJFs for tubular joints are given in the following.

### Non-dimensional influence factor expressions for local joint flexibility of single-brace joints

#### SINGLE-BRACE JOINTS

\[
\begin{align*}
\text{f}^{\text{axl}}_{\text{axl}} &= 5.69 \tau^{-0.111} \exp(-2.251\beta) \gamma^{1.898} \sin^{1.769} \theta \\
\text{f}^{\text{ipb}}_{\text{ipb}} &= 1.39 \tau^{-0.238}\beta^{-2.245} \gamma^{1.898} \sin^{1.240} \theta \\
\text{f}^{\text{opf}}_{\text{opf}} &= 55\tau^{-0.220}\exp(-4.076\beta) \gamma^{2.417} \sin^{1.893} \theta \\
\end{align*}
\]

### Non-dimensional influence factor expressions for local joint flexibility of X joints

#### CROSS JOINTS

\[
\begin{align*}
\text{f}^{\text{axl}}_{\text{axl}} &= 8.94 \tau^{-0.108} \exp(-2.759\beta) \gamma^{1.791} \sin^{1.700} \theta \\
\text{f}^{\text{ipb}}_{\text{ipb}} &= 67.60 \tau^{-0.060} \exp(-4.056\beta) \gamma^{1.897} \sin^{1.255} \theta \\
\text{f}^{\text{opf}}_{\text{opf}} &= 73.95 \tau^{-0.300} \exp(-4.478\beta) \gamma^{2.367} \sin^{1.926} \theta \\
\end{align*}
\]

### Non-dimensional influence factor expressions for local joint flexibility of K joints

#### GAPPED JOINTS

\[
\begin{align*}
\text{f}^{\text{axl}}_{\text{axl}} &= 5.90 \tau^{-0.114} \exp(-2.163\beta) \gamma^{1.899} \sin^{1.800} \theta_1 \sin^{0.000} \theta_2 \\
\text{f}^{\text{ipb}}_{\text{ipb}} &= 52.2 \tau^{-0.119} \exp(-3.835\beta) \gamma^{1.978} \sin^{0.011} \theta_1 \sin^{1.417} \theta_2 \sin^{0.108} \theta_2 \\
\text{f}^{\text{opf}}_{\text{opf}} &= 49.7 \tau^{-0.251} \exp(-4.165\beta) \gamma^{2.460} \sin^{0.004} \theta_1 \sin^{1.655} \theta_2 \sin^{0.054} \theta_2 \\
\text{f}^{\text{opb}}_{\text{opb}} &= 3.93 \tau^{-0.113} \exp(-2.198\beta) \gamma^{1.994} \sin^{0.056} \theta_1 \sin^{0.837} \theta_2 \sin^{0.794} \theta_2 \\
\end{align*}
\]

#### OVERLAPPED JOINTS

\[
\begin{align*}
\text{f}^{\text{axl}}_{\text{axl}} &= 3.91 \exp(-2.265\beta) \gamma^{2.010} \sin^{1.811} \theta_1 \sin^{0.009} \theta_2 \\
\text{f}^{\text{ipb}}_{\text{ipb}} &= 1.86 \beta^{-2.000} \gamma^{1.765} \sin^{0.029} \sin^{0.711} \theta_1 \sin^{0.006} \theta_2 \\
\text{f}^{\text{opf}}_{\text{opf}} &= 54.2 \exp(-3.959\beta) \gamma^{2.403} \sin^{0.001} \sin^{1.856} \theta_1 \sin^{0.009} \theta_2 \\
\text{f}^{\text{ipb}}_{\text{ipb}} &= 54.2 \exp(-3.959\beta) \gamma^{2.403} \sin^{0.001} \sin^{1.856} \theta_1 \sin^{0.009} \theta_2 \\
\text{f}^{\text{opf}}_{\text{opf}} &= 54.2 \exp(-3.959\beta) \gamma^{2.403} \sin^{0.001} \sin^{1.856} \theta_1 \sin^{0.009} \theta_2 \\
\text{f}^{\text{ipb}}_{\text{ipb}} &= 54.2 \exp(-3.959\beta) \gamma^{2.403} \sin^{0.001} \sin^{1.856} \theta_1 \sin^{0.009} \theta_2 \\
\end{align*}
\]

\( g \) and \( \theta_0 \) and \( \theta_1 \) = Axial Deflection and IPB and OPB Rotations

Subscripts 1 and 2 = Brace 1 and Brace 2

\( \gamma \) = Absolute value of \( g / D \)

\[ f_{\text{axl}} = \text{LJF}_{\text{axl}} \ast E \cdot D; f_{\text{ipb}} = \text{LJF}_{\text{ipb}} \ast E \cdot D^3; f_{\text{opf}} = \text{LJF}_{\text{opf}} \ast E \cdot D^3 \]
APPENDIX C  STRESS CONCENTRATION FACTORS FOR GIRTH WELDS

Reference is made to DNV-RP-C203.
APPENDIX D  STRESS EXTRAPOLATION FOR WELDS

Reference is made to DNV-RP-C203.
E.1 Stress Concentrations at Tubular Joints

E.1.1 General
High stress concentrations normally exist at the weld toe of tubular joints. The stresses may be divided into three types as shown schematically in Figure E-1:

1) The geometric stress which depends on the structural geometry of the joint
2) The notch stress, which depends on the local geometry configuration of the brace-weld-chord connection
3) The local stress at the weld toe due to the geometry of the weld bead

The geometric stress can be defined by a linear extrapolation of two stresses to the weld toe of the joint, see also App.D for definition of stress extrapolation points. Since the hot spot stress is defined by extrapolating the stresses at points A and B in Figure E-1, it is a rather arbitrary value and it will not represent the actual stress condition at the weld toes. However, the hot spot stress is a useful parameter and it is normally used for both fatigue design and for comparisons with test data for tubular joints.

The notch stress can be defined as the locally raised stress between point B and the weld toe.

The local stress at the weld toe depends on the local geometry of the weld bead, but it is independent of the joint geometry. The local stress at the weld toe quickly decays and may only be influential up to about 2 to 3 mm in depth.

The local stress concentration due to the local geometry of the weld bead may be taken into account in fracture mechanics calculations using the geometry correction factor, \( F_G \), which is given in [E.3.2].
E.2 Stresses at tubular joints

E.2.1 General

E.2.1.1 Figure E-2 shows a schematic view of the stresses which may be expected to be present at a tubular joint.

Figure E-2
Schematic view of:
- a) Stresses due to global bending moment at the joint
- b) Nominal tensile or compressive stresses
- c) Stresses due to local plate bending in chord member/wall

In Figure E-2 a, the stresses due to the global bending moment at the joint are shown. These stresses can be computed by applying simple beam theory. The stresses may be assumed constant through the thickness of the chord wall, where the fatigue crack penetrates.

E.2.1.2 When a load is applied at the top of the brace, a part of the chord wall is pulled up or pushed down to accommodate the deformation of the brace, see Figure E-2 b. It may be noted that the centre of rotation of the brace is at the intersection between the centre line of the brace and the line A-B, see Figure E-2 a. The deformation of brace results in tensile or compressive membrane stresses in the chord wall. Tensile membrane stresses arise at side A when the load acts in the direction indicated by –P in Figure E-2 a.

E.2.1.3 As illustrated in Figure E-2 c, the chord wall further deforms and local bending stresses arise in the chord wall. Typically a high percentage of the total stresses in the hot spot areas are due to this local plate bending. Hence, the degree-of-bending parameter, DoB, defined as the ratio between the bending stress and the total stress at the outer side of the chord wall, is typically 70 to 80% for tubular joints.
E.3 Stress intensity factor

E.3.1 General

The stress intensity factor for a semi-elliptical surface crack subjected to tensile membrane stress, $S_m$ and bending stress, $S_b$ can be expressed by the following semi-empirical equation,

$$K = \left( F_m \cdot S_m + F_b \cdot S_b \right) \sqrt{\pi c} \tag{E.1}$$

- $S_m$ = tensile membrane stress component
- $S_b$ = outer-fibre bending stress component
- $c$ = crack depth
- $F$ = correction factor depending on structural geometry, crack size and shape, proximity of the crack tip to free surfaces and the type of loading. Subscript “m” refers to membrane and the subscript “b” refers to bending.

It should be emphasised, that the expression in eqn. (E.1) was derived for statically determinate flat plate configurations. In the case of tubular joints, which contain some degree of redundancy, the cracked section may transfer significantly lower load as a consequence of the load shedding from the cracked section to less stressed parts of the joint, see [E.3.5].

E.3.2 Correction factor for membrane stress component

E.3.2.1 An approximate method for calculation of the stress intensity factor for a semi-elliptical crack in a welded structural detail is outlined in the following. Reference is made to Figure E-3.

![Schematic of semi-elliptical surface crack growing from weld toe](image)

**Figure E-3**
Schematic of semi-elliptical surface crack growing from weld toe

E.3.2.2 Separating the stress intensity factor into a finite number of dimensionless stress intensity factor corrections, the stress intensity factor $K$ can be expressed as follows:

$$K = F_S \cdot F_E \cdot F_T \cdot F_G \cdot S \cdot \sqrt{\pi c} \tag{E.2}$$

where $F_S$ is the (front) free surface correction factor, $F_E$ is the elliptical crack shape correction factor, $F_T$ is the finite plate thickness correction factor (or finite width correction factor), $F_G$ is the stress gradient or geometry correction factor, $S$ is the external, remote applied stress and $c$ is the physical crack length.

For a semi-elliptical crack emanating from the weld toe, see Figure E-3, the following correction factors can be applied to express the stress intensity factor corrections in eqn. (E.2).

$$F_S = 1.12 - 0.12 \cdot \frac{c}{b} \tag{E.3}$$

and

$$F_T = \sqrt{\sec(\pi c/2t)} \tag{E.4}$$

where $t$ is the thickness of the specimen.
Figure E-4
Semi-elliptical crack

The elliptical crack shape correction factor, $F_E$, is given by

$$F_E = \frac{1}{E_K} \left\{ \sin^2 \varphi + \frac{c^2}{b^2} \cos^2 \varphi \right\}^{1/4}$$ \hspace{1cm} (E.5)

in which the symbols used are explained in Figure E-4.

The value of $F_E$ is largest where the minor axis intersects the crack front (point A in Figs. 3 and 4). At this point $\varphi = \pi/2$ and eqn. (E.5) reduces to

$$F_E = \frac{1}{E_K}$$ \hspace{1cm} (E.6)

The value of $E_K$ in eqs. (X.5 and X.6) is the complete elliptical integral of the second kind, i.e.

$$E_K = \int_0^{\pi/2} \left[ 1 - \left( \frac{b^2 - c^2}{b^2} \right) \sin^2 \theta \right]^{1/2} d\theta$$ \hspace{1cm} (E.7)

which depends only upon the semi-axis ratio, $c/b$.

The value of the elliptical integral varies from $E_K = \pi/2$ for the circular crack, $c/b = 1$, to a value of $E_K = 1.0$ for the tunnel crack, as the semi-axis ratio, $c/b$, approaches zero.

A good approximation to eqn. (E.6) is obtained through the expression:

$$F_E = \left[ 1 + 4.5945 \left( \frac{c}{2b} \right)^{1.65} \right]^{-1/2}$$ \hspace{1cm} (E.8)

which also pertains to point A in Figs. 3 and 4.

The geometry correction factor, $F_G$, can be calculated applying the following formula:

$$F_G = \frac{2}{\pi} \int_0^c \frac{\sigma(x)}{\sqrt{c^2 - x^2}} dx$$ \hspace{1cm} (E.9)

where $\sigma(x)$ is the stress distribution in the un-cracked body at the line of potential crack growth due to a unit remote applied stress, and $c$ is the physical crack length. $\sigma(x)$ may, for example, be determined by a finite element calculation.

If only a finite number of stress values, $\sigma_i$ ($i = 1, 2, \ldots, n$), are known, the following equation may be used instead of eqn. (E.9)

$$F_G = \frac{2}{\pi} \sum_{i=1}^n \frac{\sigma_i}{c} \left[ \arcsin \frac{a_{i+1}}{c} - \arcsin \frac{a_i}{c} \right] (j = 1, 2, \ldots, n)$$ \hspace{1cm} (E.10)

where $(a_{i+1} - a_i)$ is the width of stress element $i$ carrying the stress $\sigma_i$ and $j$ is the number of discrete stress elements from the centre of the crack to the physical crack tip, see Figure E-5.
Since the fatigue crack in a tubular connection or joint will initiate at the weld toe as a semi-elliptical crack and finally propagate through the thickness of the chord wall, the same stress intensity factor as given in eqn. (E.1) can be applied:

\[ F_M = F_S \cdot F_E \cdot F_T \cdot F_{Gm} \]  \hfill (E.11)

where

\[ F_S = 1.12 - 0.12 \cdot \frac{c}{b} \]  \hfill (E.12)

\[ F_E = \left[ 1 + 4.5945 \left( \frac{c}{2b} \right)^{1.65} \right]^{-1/2} \]  \hfill (E.13)

\[ F_T = \sqrt{\sec(\pi c/2t)} \]  \hfill (E.14)

\( F_{Gm} \) = Geometry correction factor for the membrane stress component to be calculated according to eqn. (E.9) or eqn. (E.10).

In eqn. (E.14), \( t \) denotes the wall thickness.

### E.3.3 Correction factor for bending stress component

At the deepest point of the crack front of a semi-elliptical surface crack, the stress intensity factor correction for the bending stress component can be determined as

\[ F_b = \frac{F_{Gb}}{F_{Gm}} \cdot H \cdot F_m \]  \hfill (E.15)

where

\[ H = 1 + G_1 \cdot \left[ \frac{c}{t} \right] + G_2 \cdot \left[ \frac{c}{t} \right]^2 \]  \hfill (E.16)

\[ G_1 = -1.22 - 0.12 \cdot \frac{c}{b} \]  \hfill (E.17)

\[ G_2 = 0.55 - 1.05 \left[ \frac{c}{b} \right]^{0.75} + 0.47 \left[ \frac{c}{b} \right]^{1.5} \]  \hfill (E.18)

for \( \frac{c}{b} \leq 1 \).

In general, the geometry correction factor \( F_{Gb} \) for the bending stress component is different from \( F_{Gm} \) for the membrane stress component. \( F_{Gb} \) can be calculated from the results of a finite element analysis applying eqn. (E.9) or eqn. (E.10).

In Figure E-6, the parameter \( H \) (which is equal to the ratio between the stress intensities for bending and membrane stress components, if \( F_{Gb} = F_{Gm} \)) is plotted against the relative crack depth \( c/t \).
It appears from Figure E-6 that the reduction in $H$ for increasing crack depth is largest for the semi-circular surface crack ($c/b = 1$). In Figure E-6 it may also be seen that for high values of the semi-axis ratio $c/b$ and large relative crack depths, the parameter $H$ becomes negative and thus the bending effect may lead to a reduction in the total stress intensity and hence to lower crack growth rates.

### E.3.4 Crack shape and initial crack size

Fatigue cracks at the weld toe in tubular joints appear to be very slender with semi-axis ratios, $c/b$ less than ~0.2. The low aspect ratios for cracks in tubular joints are mainly due to crack coalescence.

Therefore, for a semi-axis ratio of $c/b = 0.2$ the initial crack size can be chosen as $c_i = 0.1$ mm

### E.3.5 Load Shedding

**E.3.5.1** The stress distribution through a tubular joint is strongly affected by the presence of a crack. As a crack is growing through the hot spot region, the load is redistributed to less stressed parts of the joint – the load shedding effect.

**E.3.5.2** A simplified model can be applied to model load shedding. By a hinge analogy the membrane stress component in the cracked section can be assumed to be unaffected by the crack, whereas the bending stress component is allowed to decrease linearly with crack depth according to the expression:

$$ S_b = S_b^\infty \left( 1 - \frac{c}{t} \right) $$

(E.19)

where $S_b^\infty$ is the bending stress component of the hot spot stress at the outer side of the chord wall in the un-cracked state. It may be noted that eqn. (E.19) has been implemented in a fracture mechanics code for crack growth analysis in weld geometries.

### E.3.6 Crack growth

**E.3.6.1** The crack growth can be calculated using the following relation

$$ \frac{dc}{dn} = C (\Delta K_{\text{m eff}} - \Delta K_{\text{m eff,th}}), \text{ for } \Delta K_{\text{eff}} \geq \Delta K_{\text{eff,th}} \quad (E.20) $$

$$ \frac{dc}{dn} = 0, \text{ for } \Delta K_{\text{eff}} < \Delta K_{\text{eff,th}} $$
For the fracture mechanics calculations the crack growth coefficients given in Table E-1 can be applied:

<table>
<thead>
<tr>
<th>Table E-1 Crack growth coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td>( m )</td>
</tr>
<tr>
<td>Mean value ( (\mu_{\log C}) )</td>
</tr>
<tr>
<td>Welds in air and in seawater with adequate corrosion protection</td>
</tr>
<tr>
<td>Welds subjected to seawater without corrosion protection</td>
</tr>
</tbody>
</table>

Here, \( \mu_{\log C} \) denotes the mean value of \( \log C \), and \( \sigma_{\log C} \) denotes the standard deviation of \( \log C \).

\[ \Delta K_{\text{eff,th}} = 79.1 \text{ MPa} \sqrt{\text{mm}} \text{ (valid in air as well as in seawater with/without corrosion protection)} \]

**E.3.6.2** The fatigue life can then be calculated by applying the method outlined above and using eqn. (E.20). For deterministic fatigue life calculations, the data tabulated for the mean + 2 standard deviations of \( \log C \) are to be applied. For probabilistic fatigue life calculations, the data tabulated for the mean value of \( \log C \) are to be applied. The fatigue life is calculated based on the through thickness crack criterion for the final crack size \( c_f \), i.e. \( c_f \sim t \), where \( t \) is the wall thickness.

**E.3.6.3** Reference is made to BS 7910 for an alternative method for fracture mechanics analyses and calculations.
APPENDIX F PILE RESISTANCE AND LOAD-DISPLACEMENT RELATIONSHIPS

F.1 Axial pile resistance

F.1.1 General

Axial pile resistance is composed of two parts

— accumulated skin resistance
— tip resistance.

For a pile in a stratified soil deposit of \( N \) soil layers, the pile resistance \( R \) can be expressed as

\[
R = R_S + R_T = \sum_{i=1}^{N} f_{S_i} A_{S_i} + q_T A_T
\]

where \( f_{S_i} \) is the average unit skin friction along the pile shaft in layer \( i \), \( A_{S_i} \) is the shaft area of the pile in layer \( i \), \( q_T \) is the unit end resistance, and \( A_T \) is the gross tip area of the pile.

F.1.2 Clay

F.1.2.1 For piles in mainly cohesive soils, the average unit skin friction \( f_{S_i} \) may be calculated according to

(1) total stress methods, e.g. the \( \alpha \) method, which yields

\[
f_{S_i} = \alpha s_u
\]

in which

\[
\alpha = \begin{cases} 
1 & \text{for } s_u/p_0'' \leq 1.0 \\
\frac{2\sqrt{s_u/p_0''}}{s_u/p_0'} & \text{for } s_u/p_0' > 1.0
\end{cases}
\]

where \( s_u \) is the undrained shear strength of the soil and \( p_0'' \) is the effective overburden pressure at the point in question.

(2) effective stress methods, e.g. the \( \beta \) method, which yields

\[
f_{S_i} = \beta p_0''
\]

in which \( \beta \) values in the range 0.10 to 0.25 are suggested for pile lengths exceeding 15 m.

(3) semi-empirical \( \lambda \) method, by which the soil deposit is taken as one single layer, for which the average skin friction is calculated as

\[
f_S = \lambda (p_{0m}'' + 2s_{um})
\]

where \( p_{0m}'' \) is the average effective overburden pressure between the pile head and the pile tip, \( s_{um} \) is the average un-drained shear strength along the pile shaft, and \( \lambda \) is the dimensionless coefficient, which depends on the pile length as shown in Figure F-1. Hence, by this method, the total shaft resistance becomes \( R_S = f_S A_S \), where \( A_S \) is the pile shaft area.

For long flexible piles, failure between pile and soil may occur close to the seabed even before the soil resistance near the pile tip has been mobilized at all. This is a result of the flexibility of the pile and the associated differences in relative pile-soil displacement along the length of the pile. This is a length effect, which for a strain-softening soil will imply that the static capacity of the pile will be less than that of a rigid pile.
For deformation and stress analysis of an axially loaded flexible pile, the pile can be modelled as a number of consecutive column elements supported by nonlinear springs applied at the nodal points between the elements. The nonlinear springs are denoted t-z curves and represent the axial load-displacement relationship between the pile and the soil. The stress $t$ is the axial skin friction per unit area of pile surface and $z$ is the relative axial pile-soil displacement necessary to mobilize this skin friction.

**F.1.2.2**  The unit tip resistance of piles in cohesive soils can be calculated as

$$q_p = N_c s_u$$

where $N_c = 9$ and $s_u$ is the undrained shear strength of the soil at the pile tip.

**F.1.3 Sand**

**F.1.3.1**  For piles in mainly cohesionless soils (sand), the average unit skin friction may be calculated according to

$$f_S = K p_0' \tan \delta f_l$$

in which $K = 0.8$ for open-ended piles and $K = 1.0$ for closed-ended piles, $p_0'$ is the effective overburden pressure, $\delta$ is the angle of soil friction on the pile wall as given in Table F-1, and $f_l$ is a limiting unit skin friction, see Table F-1 for guidance.

**F.1.3.2**  The unit tip resistance of plugged piles in cohesionless soils can be calculated as

$$q_p = N_q p_0' \leq q_l$$

in which the bearing factor $N_q$ can be taken from Table F-1 and $q_l$ is a limiting tip resistance, see Table F-1 for guidance.
The t-z curves can be generated according to a method by which a nonlinear relation applies between the origin and the point where the maximum skin resistance $t_{\text{max}}$ is reached,

$$z = t \frac{R}{G_0} \ln \left( \frac{z_{IF} - r_f}{t_{\text{max}}} \right)$$

for $0 \leq t \leq t_{\text{max}}$

in which $R$ denotes the radius of the pile, $G_0$ is the initial shear modulus of the soil, $z_{IF}$ is a dimensionless zone of influence, defined as the radius of the zone of influence around the pile divided by $R$, and $r_f$ is a curve fitting factor. For displacements $z$ beyond the displacement where $t_{\text{max}}$ is reached, the skin resistance $t$ decreases in linear manner with $z$ until a residual skin resistance $t_{\text{res}}$ is reached. For further displacements beyond this point, the skin resistance $t$ stays constant. An example of t-z curves generated according to this method is given in Figure F-2. The maximum skin resistance can be calculated according to one of the methods for prediction of unit skin friction given above.

### Table F-1  Design parameters for axial resistance of driven piles in cohesionless silicious soil

<table>
<thead>
<tr>
<th>Density</th>
<th>Soil description</th>
<th>$\delta$ (degrees)</th>
<th>$f_I$ (kPa)</th>
<th>$N_q$ (—)</th>
<th>$q_1$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose Loose Medium</td>
<td>Sand Sand-silt ²) Silt</td>
<td>15</td>
<td>48</td>
<td>8</td>
<td>1.9</td>
</tr>
<tr>
<td>Loose Medium</td>
<td>Sand Sand-silt ²) Silt</td>
<td>20</td>
<td>67</td>
<td>12</td>
<td>2.9</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>Sand Sand-silt ²) Silt</td>
<td>25</td>
<td>81</td>
<td>20</td>
<td>4.8</td>
</tr>
<tr>
<td>Dense Very dense</td>
<td>Sand Sand-silt ²)</td>
<td>30</td>
<td>96</td>
<td>40</td>
<td>9.6</td>
</tr>
<tr>
<td>Dense Very dense</td>
<td>Gravel Sand</td>
<td>35</td>
<td>115</td>
<td>50</td>
<td>12.0</td>
</tr>
</tbody>
</table>

¹) The parameters listed in this table are intended as guidelines only. Where detailed information such as in-situ cone penetrometer tests, strength tests on high quality soil samples, model tests or pile driving performance is available, other values may be justified.

²) Sand-silt includes those soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

**Figure F-2**

Example of t-z curves generated by model
For normally consolidated clays, the initial shear modulus of the soil to be used for generation of t-z curves can be taken as

\[ G_0 = \frac{300}{I_p} \cdot s_u \]

in which \( I_p \) denotes the plasticity index of the clay and is to be given as a unitless absolute number, not as a percentage. As an alternative, which is valid for both normally consolidated clays and overconsolidated clays, the following expression may be used which includes a dependency on the overconsolidation ratio,

\[ G_0 = 600 \cdot s_u - 170 \cdot s_u \sqrt{OCR - 1} \]

where \( s_u \) is the undrained shear strength of the clay, and OCR is the overconsolidation ratio. For sands, the initial shear modulus of the soil to be used for generation of t-z curves is to be taken as

\[ G_0 = \frac{m \sigma_a \sigma_v}{2(1 + \nu)} \text{ with } m = 1000 \tan \phi \]

in which \( \sigma_a = 100 \text{ kPa} \) is a reference pressure and \( \sigma_v \) is the vertical effective stress, \( \nu \) is the Poisson’s ratio of the soil, and \( \phi \) is the friction angle of the soil.

**F.2 Laterally loaded piles**

**F.2.1 General**

*F.2.1.1* The most common method for analysis of laterally loaded piles is based on the use of so-called p-y curves. The p-y curves give the relation between the integral value p of the mobilized resistance from the surrounding soil when the pile deflects a distance y laterally. The pile is modelled as a number of consecutive beam-column elements, supported by nonlinear springs applied at the nodal points between the elements. The nonlinear support springs are characterized by one p-y curve at each nodal point, see Figure F-3.

The solution of pile displacements and pile stresses in any point along the pile for any applied load at the pile head results as the solution to the differential equation of the pile

\[ EI \frac{d^4y}{dx^4} + Q_A \frac{d^2y}{dx^2} - p(y) + q = 0 \]

with

\[ EI \frac{d^3y}{dx^3} + Q_A \frac{dy}{dx} = Q_L \text{ and } EI \frac{d^2y}{dx^2} = M \]

where x denotes the position along the pile axis, y is the lateral displacement of the pile, EI is the flexural rigidity of the pile, \( Q_A \) is the axial force in the pile, \( Q_L \) is the lateral force in the pile, \( p(y) \) is the lateral soil reaction, \( q \) is a distributed load along the pile, and \( M \) is the bending moment in the pile, all at the position x.
A finite difference method usually forms the most feasible approach to achieve the sought-after solution of the differential equation of the pile. A number of commercial computer programs are available for this purpose. These programs usually provide full solutions of pile stresses and displacements for a combination of axial force, lateral force and bending moment at the pile head, i.e., also the gradual transfer of axial load to the soil along the pile according to the t-z curve approach presented above is included. Some of the available programs can be used to analyse not only single piles but also pile groups, including possible pile-soil-pile interaction and allowing for proper representation of a superstructure attached at the pile heads, either as a rigid cap or as a structure of finite stiffness.

For construction of p-y curves, the type of soil, the type of loading, the remoulding due to pile installation and the effect of scour should be considered. A recommended method for construction of p-y curves is presented in the following:

The lateral resistance per unit length of pile for a lateral pile deflection \( y \) is denoted \( p \). The static ultimate lateral resistance per unit length is denoted \( p_u \). This is the maximum value that \( p \) can take on when the pile is deflected laterally.

**F.2.2 Clay**

For piles in cohesive soils, the static ultimate lateral resistance is recommended to be calculated as

\[
p_u = \begin{cases} 
(3s_u + \gamma' X)D + Js_u X & \text{for } 0 < X \leq X_R \\
9s_u D & \text{for } X > X_R
\end{cases}
\]

where \( X \) is the depth below soil surface and \( X_R \) is a transition depth, below which the value of \((3s_u + \gamma' X)D + Js_u X\) exceeds \(9s_u D\). Further, \( D \) is the pile diameter, \( s_u \) is the undrained shear strength of the soil, \( \gamma' \) is the effective unit weight of soil, and \( J \) is a dimensionless empirical constant whose value is in the range 0.25 to 0.50 with 0.50 recommended for soft normally consolidated clay.

For static loading, the p-y curve can be generated according to
For cyclic loading and \( X > X_R \), the p-y curve can be generated according to

\[
p = \begin{cases} 
\frac{p_u}{2} \left( \frac{y}{y_c} \right)^{1/3} & \text{for } y \leq 8y_c \\
0.72p_u & \text{for } y > 8y_c
\end{cases}
\]

For cyclic loading and \( X \leq X_R \), the p-y curve can be generated according to

\[
p = \begin{cases} 
\frac{p_u}{2} \left( \frac{y}{y_c} \right)^{1/3} & \text{for } y \leq 3y_c \\
0.72p_u (1 - (1 - \frac{X}{X_R}) \frac{y - 3y_c}{12y_c}) & \text{for } 3y_c < y \leq 15y_c \\
0.72p_u \frac{X}{X_R} & \text{for } y > 15y_c
\end{cases}
\]

Here, \( y_c = 2.5\varepsilon_c D \), in which \( D \) is the pile diameter and \( \varepsilon_c \) is the strain which occurs at one-half the maximum stress in laboratory undrained compression tests of undisturbed soil samples. For further details, reference is made to Classification Notes No. 30.4.

**F.2.3 Sand**

For piles in cohesionless soils, the static ultimate lateral resistance is recommended to be calculated as

\[
p_u = \begin{cases} 
(C_1 X + C_2 D)\gamma' X & \text{for } 0 < X \leq X_R \\
C_3 D\gamma' X & \text{for } X > X_R
\end{cases}
\]

where the coefficients \( C_1, C_2 \) and \( C_3 \) depend on the friction angle \( \phi \) as shown in Figure F-4, and where \( X \) is the depth below soil surface and \( X_R \) is a transition depth, below which the value of \((C_1 X + C_2 D)\gamma' X\) exceeds \( C_3 D\gamma' X \). Further, \( D \) is the pile diameter, and \( \gamma' \) is the submerged unit weight of soil.

The p-y curve can be generated according to

\[
p = A_p u \tanh \left( \frac{kX}{A_p u} y \right)
\]

![Figure F-4](image)

**Figure F-4**

Coefficients as functions of friction angle

in which \( k \) is the initial modulus of subgrade reaction and depends on the friction angle \( \phi \) as given in Figure F-
A is a factor to account for static or cyclic loading conditions as follows:

\[ A = \begin{cases} 
0.9 & \text{for cyclic loading} \\
(3 - 0.8 \frac{X}{D}) \geq 0.9 & \text{for static loading}
\end{cases} \]

For further details, reference is made to Classification Notes No. 30.4.

**Figure F-5**
Initial modulus of subgrade reaction k as function of friction angle \( \phi \)

**F.2.4 Application of p-y curves**

**F.2.4.1** The nonlinear p-y curves recommended in [F.2.2] and [F.2.2] are meant primarily for analysis of piles for evaluation of lateral pile capacity in the ULS. These p-y curves have been calibrated for long slender jacket piles with diameters of up to 1.0 m. They have not been calibrated for monopiles with larger diameters and are in general not valid for such monopiles. P-y curves to be used for monopile design should be validated for such use, e.g. by FE analysis.

**F.2.4.2** Caution must be exercised when the recommended nonlinear p-y curves are used in other contexts than for evaluation of lateral pile capacity in the ULS. Such contexts include, but are not limited to, SLS analysis of the pile, fatigue analysis of the pile, determination of equivalent spring stiffnesses to represent the stiffness of the pile-soil system as boundary condition in analyses of the structure that the pile-soil system supports, and in general all cases where the initial slope of the p-y curves may have an impact.

**F.2.4.3** Caution must be exercised regardless of whether the recommended nonlinear p-y curves are applied directly as they are specified on closed form or whether piece-wise linear approximations according to some discretisation of the curves are applied.

**F.2.4.4** The p-y curves that are recommended for clay are defined as 3rd order polynomials such that they have infinite initial slopes, i.e. the initial stiffnesses of the load-displacement relationships are infinite. This is unphysical; however, the curves are still valid for use for their primary purpose, viz. evaluation of lateral pile capacity in the ULS. However, the closed-form p-y curves that are recommended for clay cannot be used directly in cases where the initial stiffness matters, such as for determination of equivalent pile head stiffnesses.

**F.2.4.5** When a p-y curve for clay is to be used in contexts where the initial slope of the curve matters, the curve need to be discretised and approximated by a piece-wise linear curve drawn between the discretisation points. The discretisation must be carried out in such a manner that the first discretisation point of the curve beyond the origin is localised such that a correct initial slope results in the piece-wise linear representation of the p-y curve.

**F.2.4.6** Unless data indicate otherwise, the true initial slope of a p-y curve in clay may be calculated as

\[ k = \xi \frac{P_u}{D \cdot (\varepsilon_c)^{0.25}} \]

where \( \xi \) is an empirical coefficient and \( \varepsilon_c \) is the vertical strain at one-half the maximum principal stress.
difference in a static undrained triaxial compression tests on an undisturbed soil sample. For normally consolidated clay $\xi = 10$ is recommended, and for over-consolidated clay $\xi = 30$ is recommended.

**F.2.4.7** As an alternative to localising the first discretisation point beyond the origin such that a correct initial slope results in the piece-wise linear approximation of the p-y curve for clay, the first discretisation point beyond the origin may be localised at the relative displacement $y/y_c = 0.1$ with ordinate value $p/p_u = 0.23$.

**F.2.4.8** The recommended closed form p-y curves for sand have finite initial slopes and thus finite initial stiffnesses. Whenever discretised approximations to these curves are needed in analyses with piece-wise linear curves drawn through the discretisation points, it is important to impose a sufficiently fine discretisation near the origin of the p-y curves in order to get a correct representation of the initial slopes.

**F.2.4.9** Whenever p-y curves are used to establish equivalent pile head stiffnesses to be applied as boundary conditions for analysis of structures supported by a pile-soil system, it is recommended that a sensitivity study be carried out to investigate the effect of changes in or different assumptions for the initial slopes of the p-y curves.
APPENDIX G BEARING CAPACITY AND STIFFNESS FORMULAE FOR GRAVITY BASE FOUNDATIONS

G.1 Forces
All forces acting on the foundation, including forces transferred from the wind turbine, are transferred to the foundation base and combined into resultant forces H and V in the horizontal and vertical direction, respectively, at the foundation-soil interface.

\[ e = \frac{M_d}{V_d} \]

where \( M_d \) denotes the resulting design overturning moment about the foundation-soil interface.

G.2 Correction for torque
When a design torque \( M_{zd} \) is applied to the foundation in addition to the forces \( H_d \) and \( V_d \), the interaction between the torque and these forces can be accounted for by replacing \( H_d \) and \( M_{zd} \) with an equivalent horizontal force \( H'_d \). The bearing capacity of the foundation is then to be evaluated for the force set \((H'_d, V_d)\) instead of the force set \((H_d, V_d)\). The equivalent horizontal force can be calculated as:

\[ H'_d = \frac{2 \cdot M_{zd}}{l_{eff}} + \sqrt{H_d^2 + \left(\frac{2 \cdot M_{zd}}{l_{eff}}\right)^2} \]

in which \( l_{eff} \) is the length of the effective area as determined in [G.3].

G.3 Effective foundation area
For use in bearing capacity analysis an effective foundation area \( A_{eff} \) is needed. The effective foundation area is constructed such that its geometrical centre coincides with the load centre, and such that it follows as closely as possible the nearest contour of the true area of the foundation base. For a quadratic area of width \( b \), the effective area \( A_{eff} \) can be defined as

\[ A_{eff} = b_{eff} \cdot l_{eff} \]
in which the effective dimensions \( b_{\text{eff}} \) and \( l_{\text{eff}} \) depend on which of two idealised loading scenarios leads to the most critical bearing capacity for the actual foundation.

Figure G-2
Quadratic footing with two approaches to how to make up the effective foundation area

Scenario 1 corresponds to load eccentricity with respect to one of the two symmetry axes of the foundation. By this scenario, the following effective dimensions are used:

\[
b_{\text{eff}} = b - 2 \cdot e, \quad l_{\text{eff}} = b
\]

Scenario 2 corresponds to load eccentricity with respect to both symmetry axes of the foundation. By this scenario, the following effective dimensions are used:

\[
b_{\text{eff}} = l_{\text{eff}} = b - e\sqrt{2}
\]

Reference is made to Figure G-2. The effective area representation that leads to the poorest or most critical result for the bearing capacity of the foundation is the effective area representation to be chosen.

For a circular foundation area with radius \( R \), the effective foundation area \( A_{\text{eff}} \) can be defined as

\[
A_{\text{eff}} = 2 \left[ R^2 \arccos\left(\frac{e}{R}\right) - e\sqrt{R^2 - e^2} \right]
\]

This is recognized as the area of a circle segment and its mirrored image with the midpoint of their common secant located in the load application point, see Figure G-3. The width of this double circle segment area is

\[
b_c = 2(R - e)
\]

and the length is

\[
l_c = 2R \sqrt{1 - \left(1 - \frac{b_c}{2R}\right)^2}
\]
Based on this, the effective foundation area $A_{eff}$ can now be represented by a rectangle with the following dimensions:

$$l_{eff} = \sqrt{\frac{A_{eff}}{b_{eff}}} \quad \text{and} \quad b_{eff} = \frac{l_{eff}}{l_{e}}$$

For an area shaped as a double symmetrical polygon (octagonal or more), the above formulae for the circular foundation area can be used provided that a radius equal to the radius of the inscribed circle of the polygon is used for the calculations.

**G.4 Bearing capacity**

**G.4.1 General**

*G.4.1.1* For fully drained conditions and failure according to Rupture 1 as indicated in Figure G-1, the following general formula can be applied for the bearing capacity of a foundation with a horizontal base, resting on the soil surface:

$$q_d = \frac{1}{2} \gamma' b_{eff} N_c^0 \gamma' + p_0 N_q^0 q_{i}^0 + c_d N_c^0 s_{e}^i c_{i}$$

For undrained conditions, which imply $\phi = 0$, the following formula for the bearing capacity applies:

$$q_d = s_{ud}^i N_c^0 \cdot s_{e}^i c_{i} + p_0$$

The symbols used have the following explanations:

- $q_d$: design bearing capacity [kN/m$^2$]
- $\gamma'$: effective (submerged) unit weight of soil [kN/m$^3$]
- $p_0$: effective overburden pressure at the level of the foundation-soil interface [kN/m$^2$]
- $c_d$: design cohesion assessed on the basis of the actual shear strength profile, load configuration and estimated depth of potential failure surface [kN/m$^2$]
- $s_{ud}$: design undrained strength assessed on the basis of the actual shear strength profile, load configuration and estimated depth of potential failure surface [kN/m$^2$]
- $N_q$, $N_c$: bearing capacity factors, dimensionless
- $s_{y}$, $s_{s}$, $s_{e}$: shape factors, dimensionless
- $i_y$, $i_q$, $i_c$: inclination factors, dimensionless

*G.4.1.2* In principle, the quoted formulae apply to foundations, which are not embedded. However, the formulae may also be applied to embedded foundations, for which they will lead to results, which will be on
the conservative side. Alternatively, depth effects associated with embedded foundations can be calculated according to formulae given in DNV Classification Notes No. 30.4.

The calculations are to be based on design shear strength parameters:

\[ s_w = \frac{s_a}{\gamma_c}, \quad c_d = \frac{c}{\gamma_c} \quad \text{and} \quad \phi_d = \arctan \left( \frac{\tan(\phi)}{\gamma_d} \right) \]

The material factors \( \gamma_c \) an \( \gamma_d \) must be those associated with the actual design code and the type of analysis, i.e. whether drained or undrained conditions apply.

The dimensionless factors \( N \), \( s \) and \( i \) can be determined by means of formulae given in the following.

**G4.2 Bearing capacity formulae for drained conditions**

Bearing capacity factors \( N \):

\[ N_q = e^{x \tan \phi_d} \left( 1 + \frac{\sin \phi_d}{1 - \sin \phi_d} \right) \quad N_c = (N_q - 1) \cdot \cot \phi_d \quad N_\gamma = \frac{3}{2} (N_q - 1) \cdot \tan \phi_d \]

When the bearing capacity formulae are used to predict soil reaction stresses on foundation structures for design of such structures, it is recommended that the factor \( N_\gamma \) is calculated according to the following formula

\[ N_\gamma = 2 \cdot (N_q + 1) \cdot \tan \phi_d \]

Shape factors \( s \):

\[ s_p = 1 - 0.4 \cdot \frac{b_{eff}}{l_{eff}} \quad s_q = s_c = 1 + 0.2 \cdot \frac{b_{eff}}{l_{eff}} \]

Inclination factors \( i \):

\[ i_q = i_c = \left( 1 - \frac{H_d}{V_d + A_{eff} \cdot c_d \cdot \cot \phi_d} \right)^2 \quad i_\gamma = i_q^2 \]

**G4.3 Bearing capacity formulae for undrained conditions, \( \phi = 0 \)**

\[ N_c^0 = \pi + 2 \]

\[ s_c^0 = s_c \]

\[ i_c^0 = 0.5 + 0.5 \cdot \sqrt{1 - \frac{H_d}{A_{eff} \cdot s_{ud}}} \]

**G5 Extremely eccentric loading**

In the case of extremely eccentric loading, i.e., an eccentricity in excess of 0.3 times the foundation width, \( e > 0.3 \cdot b \), an additional bearing capacity calculation needs to be carried out, corresponding to the possibility of a failure according to Rupture 2 in Figure G-1. This failure mode involves failure of the soil also under the unloaded part of the foundation area, i.e., under the heel of the foundation. For this failure mode, the following formula for the bearing capacity applies

\[ q_d = y \cdot b_{eff} \cdot N_q \cdot s_i \cdot i_q + c_d \cdot N_c \cdot s \cdot i_c \cdot (1.05 + \tan^3 \phi_d) \]

with inclination factors.
The bearing capacity is to be taken as the smallest of the values for $q_d$ resulting from the calculations for Rupture 1 and Rupture 2.

### G.6 Sliding resistance

Foundations subjected to horizontal loading must also be investigated for sufficient sliding resistance. The following criterion applies in the case of drained conditions:

$$H_d < r \cdot (A_{\text{eff}} \cdot c_d + V_d \cdot \tan \phi_d)$$

For undrained conditions in clay, $\phi = 0$, the following criterion applies:

$$H_d < A_{\text{eff}} \cdot r \cdot s_{\text{ud}}$$

In either case, $r$ is a roughness parameter which is 1.0 for soil against soil and which can take on values less than 1.0 for soil against structure.

### G.7 Foundation stiffness

When the soil conditions are fairly homogeneous and an equivalent dynamic 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 Table G-1 and Table G-2. Foundation springs based on these formulae will be representative for the dynamic foundation stiffnesses that are needed in structural analyses for wind and wave loading on the wind turbine and its support structure. In structural analyses for earthquake loads, however, it may be necessary to apply frequency-dependent reductions of the stiffnesses from Table G-1 and Table G-2 to get appropriate dynamic stiffness values for the analyses. Owing to the rate of stress cycling in clay caused by dynamic wind turbine loading, the soil volume can be assumed to remain constant in the case of clay soils and Poisson’s ratio can be assumed to be $\nu = 0.5$ for the clays.
## Table G-1  Circular footing on stratum over bedrock or on stratum over half space

<table>
<thead>
<tr>
<th>Mode of motion</th>
<th>On stratum over bedrock</th>
<th>On stratum over half space</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical</td>
<td>$K_v = \frac{4GR}{1-\nu}(1+1.28\frac{R}{H})$</td>
<td>$K_v = \frac{4G_v R}{1-\nu_v}(1+1.28\frac{R}{H})$; $1 H/R \leq 5$</td>
</tr>
<tr>
<td>Horizontal</td>
<td>$K_h = \frac{8GR}{2-\nu}(1+\frac{R}{2H})$</td>
<td>$K_h = \frac{8G_v R}{2-\nu_v}(1+\frac{R}{2H})$; $1 H/R \leq 4$</td>
</tr>
<tr>
<td>Rocking</td>
<td>$K_s = \frac{8GR_1^1}{3(1-\nu)}(1+\frac{R}{6H})$</td>
<td>$K_s = \frac{8G_v R_1^1}{3(1-\nu_v)}(1+\frac{R}{6H})$; $0.75 H/R \leq 2$</td>
</tr>
<tr>
<td>Torsion</td>
<td>$K_T = \frac{16GR^3}{3}$</td>
<td>Not given</td>
</tr>
</tbody>
</table>
### Table G-2  Circular footing embedded in stratum over bedrock

<table>
<thead>
<tr>
<th>Mode of motion</th>
<th>Foundation stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Vertical</strong></td>
<td>( K_v = \frac{4GR}{1-\nu} (1+1.28 \frac{R}{H})(1+\frac{D}{2R})(1+(0.85-0.28 \frac{D}{R})\frac{D}{H}) )</td>
</tr>
<tr>
<td><strong>Horizontal</strong></td>
<td>( K_v = \frac{8GR}{2-\nu} (1+\frac{R}{2H})(1+\frac{2D}{3R})(1+\frac{5D}{4H}) )</td>
</tr>
<tr>
<td><strong>Rocking</strong></td>
<td>( K_v = \frac{8GR^3}{3(1-\nu)} (1+\frac{R}{6H})(1+2\frac{D}{R})(1+0.7 \frac{D}{H}) )</td>
</tr>
<tr>
<td><strong>Torsion</strong></td>
<td>( K_v = \frac{16GR^3}{3} (1+\frac{8D}{3R}) )</td>
</tr>
</tbody>
</table>

Range of validity:  
- \( D/R<2 \)
- \( D/H<\frac{1}{2} \)
APPENDIX H  CROSS SECTION TYPES

H.1 Cross section types

H.1.1 General

*H.1.1.1* Cross sections of beams are divided into different types dependent of their ability to develop plastic hinges as given in [Table H-1](#).

<table>
<thead>
<tr>
<th>Table H-1 Cross sectional types</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
</tr>
<tr>
<td>II</td>
</tr>
<tr>
<td>III</td>
</tr>
<tr>
<td>IV</td>
</tr>
</tbody>
</table>

![Figure H-1](#)

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

*H.1.1.2* The categorisation of cross sections depends on the proportions of each of its compression elements, see [Table H-3](#).

*H.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.

*H.1.1.4* The various compression elements in a cross section such as web or flange, can be in different classes.

*H.1.1.5* The selection of cross sectional type is normally quoted by the highest or less favourable type of its compression elements.

**H.1.2 Cross section requirements for plastic analysis**

*H.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.

*H.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.
H.1.3 Cross section requirements when elastic global analysis is used

H.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.

H.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>
<thead>
<tr>
<th>NV Steel grade&lt;sup&gt;1)&lt;/sup&gt;</th>
<th>$\varepsilon$&lt;sup&gt;2)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>NV-NS</td>
<td>1</td>
</tr>
<tr>
<td>NV-27</td>
<td>0.94</td>
</tr>
<tr>
<td>NV-32</td>
<td>0.86</td>
</tr>
<tr>
<td>NV-36</td>
<td>0.81</td>
</tr>
<tr>
<td>NV-40</td>
<td>0.78</td>
</tr>
<tr>
<td>NV-420</td>
<td>0.75</td>
</tr>
<tr>
<td>NV-460</td>
<td>0.72</td>
</tr>
<tr>
<td>NV-500</td>
<td>0.69</td>
</tr>
<tr>
<td>NV-550</td>
<td>0.65</td>
</tr>
<tr>
<td>NV-620</td>
<td>0.62</td>
</tr>
<tr>
<td>NV-690</td>
<td>0.58</td>
</tr>
</tbody>
</table>

1) The table is not valid for steel with improved weldability. See Sec.6, Table 6-3, footnote 1).

2) $\varepsilon = \frac{235}{f_y}$ where $f_y$ is yield strength in N/mm$^2$
### Table H-3 Maximum width-to-thickness ratios for compression elements

<table>
<thead>
<tr>
<th>Cross section part</th>
<th>Type I</th>
<th>Type II</th>
<th>Type III</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Diagram" /></td>
<td><img src="image2" alt="Diagram" /></td>
<td><img src="image3" alt="Diagram" /></td>
<td><img src="image4" alt="Diagram" /></td>
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<td><img src="image5" alt="Diagram" /></td>
<td><img src="image6" alt="Diagram" /></td>
<td><img src="image7" alt="Diagram" /></td>
<td><img src="image8" alt="Diagram" /></td>
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<td><img src="image9" alt="Diagram" /></td>
<td><img src="image10" alt="Diagram" /></td>
<td><img src="image11" alt="Diagram" /></td>
<td><img src="image12" alt="Diagram" /></td>
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<tr>
<td><img src="image13" alt="Diagram" /></td>
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<td><img src="image15" alt="Diagram" /></td>
<td><img src="image16" alt="Diagram" /></td>
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<td><img src="image17" alt="Diagram" /></td>
<td><img src="image18" alt="Diagram" /></td>
<td><img src="image19" alt="Diagram" /></td>
<td><img src="image20" alt="Diagram" /></td>
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<tr>
<td><img src="image21" alt="Diagram" /></td>
<td><img src="image22" alt="Diagram" /></td>
<td><img src="image23" alt="Diagram" /></td>
<td><img src="image24" alt="Diagram" /></td>
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<tr>
<td><img src="image25" alt="Diagram" /></td>
<td><img src="image26" alt="Diagram" /></td>
<td><img src="image27" alt="Diagram" /></td>
<td><img src="image28" alt="Diagram" /></td>
</tr>
<tr>
<td><img src="image29" alt="Diagram" /></td>
<td><img src="image30" alt="Diagram" /></td>
<td><img src="image31" alt="Diagram" /></td>
<td><img src="image32" alt="Diagram" /></td>
</tr>
</tbody>
</table>

1. Compression negative
2. \( \varepsilon \) is defined in Table H-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 \( k_\sigma \))
5. Valid for axial and bending, not external pressure.
APPENDIX I  EXTREME WIND SPEED EVENTS

Reference is made to [3.2.4].
APPENDIX J  SCOUR AT A VERTICAL PILE

J.1  Flow around a vertical pile

J.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 J-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 J-1
Flow around the base of a vertical pile

J.2  Bed shear stress

J.2.1  General

J.2.1.1  The increase in the bed shear stress can be expressed in terms of the amplification factor $\alpha$, which is defined by

$$\alpha = \frac{\tau_{\text{max}}}{\tau_{\text{max},\infty}} \quad (J.1)$$

in which $\tau_{\text{max}}$ is the maximum value of the bed shear stress $\tau$ when the pile structure is present and $\tau_{\text{max},\infty}$ is the maximum value of the bed shear stress $\tau_\infty$ for the undisturbed flow. In the case of a steady current, $\tau_{\text{max}}$ and $\tau_{\text{max},\infty}$ are replaced by constant $\tau$ and $\tau_\infty$, respectively, in the expression for $\alpha$.

J.2.1.2  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.

J.3  Local scour

J.3.1  General

J.3.1.1  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.

J.3.1.2  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.

J.3.1.3 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. The shape of the scour hole will virtually be symmetrical, see Figure J-2.

![Figure J-2](image)

**Figure J-2**

**Scour hole around a vertical pile**

J.3.1.4 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

\[ KC = \frac{u_{\text{max}} \cdot T}{D} \] (J.2)

where \( T \) is the wave period, \( D \) is the cylinder diameter and \( u_{\text{max}} \) is the maximum value of the orbital velocity at the bed, given by linear theory as:

\[ u_{\text{max}} = \frac{\pi \cdot H}{T \sinh(kh)} \] (J.3)

Here \( H \) is the wave height, \( h \) is the water depth and \( k \) is the wave number which can be found by solving the dispersion equation:

\[ \left( \frac{2\pi}{T} \right)^2 = g \cdot k \tanh(kh) \] (J.4)

where \( g \) denotes the acceleration of gravity, i.e. 9.81 m/s².

**J.3.2 Scour depth**

Unless data, e.g. from model tests, indicate otherwise, the following empirical expression for the equilibrium scour depth \( S \) may be used:

\[ \frac{S}{D} = 1.3 \left[ 1 - \exp \left( -0.03(KC - 6) \right) \right] \quad KC \geq 6 \] (J.5)

Caution must be exercised when using this expression, in particular for large-diameter cylinders such as monopiles. The expression is valid for live-bed conditions, i.e. for \( \theta > \theta_{\text{cr}} \), in which the Shields parameter \( \theta \) is defined below together with its critical threshold \( \theta_{\text{cr}} \). For steady current, which implies \( KC \rightarrow \infty \), it appears from this expression that \( S/D \rightarrow 1.3 \). 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 \( KC < 6 \). The Shields parameter \( \theta \) is defined by:

\[ \theta = \frac{U_{\text{f}}^2}{g(s-1)d} \] (J.6)

where \( s \) is the specific gravity of the sediment, \( d \) is the grain diameter for the specific grain that will be eroded and \( U_{\text{f}} \) 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, \( \theta_{\text{cr}} \), is the value of \( \theta \) at the initiation of sediment motion. The critical value \( \theta_{\text{cr}} \) for the Shields parameter is about 0.05 to 0.06. Seabed erosion starts when the Shields parameter exceeds the critical value.

For steady current the bed shear velocity, \( U_{\text{f}} \), is given by the Colebrook and White equation

\[ \frac{U_{\text{f}}}{U_{\text{f}}^*} = 6.4 - 2.5 \cdot \ln \left( \frac{2.5 \cdot d}{h} + \frac{4.7 \cdot v}{h \cdot U_{\text{f}}^*} \right) \] (J.7)
where \( \nu = 10^{-6} \text{m}^2/\text{s} \) is the kinematic viscosity. For waves, the maximum value of the undisturbed bed shear velocity is calculated by:

\[
U_f = \sqrt{\frac{f_w}{2}} \cdot u_{\text{max}} \quad (J.8)
\]

where \( f_w \) is the frictional coefficient given by

\[
f_w = \begin{cases} 
0.04 \cdot (a / k_N)^{-0.25} & a / k_N > 100 \\
0.4 \cdot (a / k_N)^{-0.75} & a / k_N < 100
\end{cases} \quad (J.9)
\]

Here, \( a \) is the free stream amplitude, defined by

\[
a = \frac{u_{\text{max}} \cdot T}{2\pi} \quad (J.10)
\]

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 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.

### J.3.3 Lateral extension of scour hole

The scour depth \( S \) is estimated by means of the empirical expression in eqn. (J.5), which is valid for live bed conditions. The lateral extension of the scour hole at the original level of the seabed can be estimated based on the friction angle \( \phi \) of the soil, and assuming that the slope of the scour hole equals this friction angle. By this approach, the radius of the scour hole, measured at the original level of the seabed from the centre of a pile of diameter \( D \), is estimated as

\[
r = \frac{D}{2} + \frac{S}{\tan \phi} \quad (J.11)
\]

### J.3.4 Time scale of scour

The temporal evolution of the scour depth, \( S \), can be expressed as:

\[
S_t = S (1 - \exp(-t / T_1)) \quad (J.12)
\]

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

\[
T^* = \frac{\sqrt{g(s-1)d^3}}{D^2} \cdot T_1 \quad (J.13)
\]

where \( T^* \) is given by the empirical expressions:

\[
T^* = \frac{1}{2000} \cdot \frac{h}{D} \cdot \theta^{-2.2} \quad \text{for steady current} \quad (J.14)
\]

\[
T^* = 10^{-6} \cdot \left( \frac{KC}{\theta} \right)^3 \quad \text{for waves} \quad (J.15)
\]
APPENDIX K CALCULATIONS BY FINITE ELEMENT METHOD

K.1 Introduction

K.1.1 General

K.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.

K.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.

K.1.1.3 Since a FEM analysis is normally used when simple calculations are insufficient or impossible, care must be taken to ensure that the model and analysis reflect the physical reality. This must be done by means of carrying out an evaluation of the input to as well as the results from the analysis. Guidelines for such an evaluation are given below.

K.2 Types of analysis

K.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.

K.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. As 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.

K.2.3 Frequency analysis

K.2.3.1 Frequency analysis is used to determine the eigenfrequencies and normal modes of a structural part.

K.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.

---end---of---Guidance---note---
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.

---end---of---Guidance---note---

K.2.7 Other types of analyses

K.2.7.1 The analyses listed in [K.2.2] through [K.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.

K.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.

K.3 Modelling

K.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.

K.3.2 Model

The input for an FEM model must be documented thoroughly by relevant printouts and plots. The printed data should preferably be stored or supplied as files on a CD-ROM.

K.3.3 Coordinate systems

K.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.

K.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.

K.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.

K.3.4 Material properties

K.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.

K.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).

K.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.

K.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.

K.3.7 Element types

K.3.7.1 1D elements consist of beam elements.

Models with beam elements are quite simple to create and provide good results for framework 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.

K.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.

K.3.7.3 The stresses at connections such as welds cannot be calculated directly by these elements either.

K.3.7.4 3D elements consist of solid elements.

K.3.7.5 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 solution time will 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.

K.3.8 Combinations

K.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.

K.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.

K.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.

K.3.9 Element size and distribution of elements

K.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.

K.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.

K.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.

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 K-1.

<table>
<thead>
<tr>
<th>E = 2.1 \times 10^5 \text{ N/mm}^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 N</td>
</tr>
<tr>
<td>10 mm</td>
</tr>
<tr>
<td>100 mm</td>
</tr>
</tbody>
</table>

Figure K-1
Cantilever

Table K-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.
K.3.10 Element quality

K.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.

K.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.

K.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.

K.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.

K.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.

K.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.

K.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.

K.3.11 Boundary conditions

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.

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.
K.3.12 Types of restraints

K.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.

K.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 result in singularities.

An appropriate value for the stiffness of such a stiff spring may be approximately $10^6$ times the largest stiffness of the model.

K.3.12.3 As the program must first identify whether the displacement has to be constrained or free, the contact boundary condition requires a non-linear calculation.

K.3.13 Symmetry/antimetry

K.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.

K.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.

K.3.13.3 The loads for a symmetric model may be a combination of a symmetric and an antimetric load. 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.

K.3.13.4 If both model and loads have rotational symmetry, a sectional model is sufficient for calculating the response.

K.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.

K.3.14 Loads

K.3.14.1 The loads applied for the FEM calculation are usually structural loads, however, centrifugal loads and temperature loads are also relevant.

K.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.

K.3.15 Load application

K.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.

K.3.15.2 To circumvent the problems involved with execution of an entirely new calculation when only a slightly 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.

K.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.
K.4 Documentation

K.4.1 Model

K.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.

K.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.

K.4.1.3 The model aspects listed in [K.4.2] through [K.4.7] can and should be checked prior to execution of the FEM analysis.

K.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.

K.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.

K.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.

K.4.5 Element type

Several different element types can be used, and here plots and listing of the element types should also be presented.

K.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.

K.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.

K.4.8 Reactions

K.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.

K.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.

K.4.9 Mesh refinement

The simplest way of establishing whether the present model or mesh is dense enough is to remesh 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.

K.4.10 Results

K.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.

K.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.

K.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.

K.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.

K.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.

K.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.

K.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.
APPENDIX L  ICE LOADS FOR CONICAL STRUCTURES

L.1 Calculation of ice loads

Calculation of ice loads on conical structures such as ice cones in the splash zone of monopiles and gravity base structures can be carried out by application of Ralston’s formulae, which are based on plastic limit analysis.

Ralston’s formulae distinguish between upward breaking cones and downward breaking cones, see Figure L-1. For offshore wind turbine structures, downward breaking cones are most common.

![Figure L-1](image_url)

Upward breaking cone (left) and downward breaking cone (right)

For upward breaking cones, the horizontal force on the cone is

\[ R_H = (A_1 \sigma f h^2 + A_2 \gamma_w h b^2 + A_3 \gamma_w h (b^2 - b_T^2)) A_4 \]

The vertical force on the cone is

\[ R_v = B_1 R_H + B_2 \gamma_w h (b^2 - b_T^2) \]

For downward breaking cones, also known as inverted cones, the horizontal force on the cone is

\[ R_H = (A_1 \sigma f h^2 + \frac{1}{9} A_2 \gamma_w h b^2 + \frac{1}{9} A_3 \gamma_w h (b^2 - b_T^2)) A_4 \]

The vertical force on the cone is

\[ R_v = B_1 R_H + \frac{1}{9} B_2 \gamma_w h (b^2 - b_T^2) \]

The following symbols are used in these expressions

- \( \sigma_f \) = flexural strength of ice
- \( \gamma_w \) = specific weight of seawater
- \( h \) = ice sheet thickness
- \( b \) = cone diameter at the water line
- \( b_T \) = cone diameter at top of cone

\( A_1, A_2, A_3, A_4, B_1 \) and \( B_2 \) are dimensionless coefficients, whose values are functions of the ice-to-cone friction coefficient \( \mu \) and of the inclination angle \( \alpha \) of the cone with the horizontal. Graphs for determination of the coefficients are given in Figure L-2.

The argument \( k \) is used for determination of the coefficients \( A_1 \) and \( A_2 \) from Figure L-2.

For upward breaking cones,

\[ k = \frac{\gamma_w h^2}{\sigma_f h} \]

shall be used.
For downward breaking cones,

\[ k = \frac{\gamma_s b^2}{9\sigma/h} \]

shall be used.  

The inclination angle \( \alpha \) with the horizontal should not exceed approximately 65° in order for the theories underlying the formulae to be valid.

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**Figure L-2**

Ice force coefficients for plastic limit analysis according to Ralston’s formulae
CHANGES – HISTORIC

Note that historic changes older than the editions shown below have not been included. Older historic changes (if any) may be retrieved through http://www.dnv.com.

February 2013 edition

Amendment February 2013

— The date in the page header has been corrected from January 2012 to January 2013.

Main changes

• Sec.8 Detailed Design of Offshore Concrete Structures

— B100 Table B1: Changes in material factor requirements for concrete design to bring the requirements in alignment with changed expressions for concrete strength in DNV-OS-C502.