STANDARD

DNVGL-ST-0119 Edition July 2018

Floating wind turbine structures

DNV GL AS

FOREWORD

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

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

CHANGES - CURRENT

This document supersedes the June 2013 edition of DNV-OS-J103.

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

Changes July 2018

Topic	*Reference*	*Description*
New title		The previous title of DNV-OS-J103 has been changed.
Change of terminology	Whole document	The term ship impact has been replaced by boat impact to align with DNVGL-ST-0437, DNVGL-ST-0126 and IEC 61400-3-1.
Section restructured	Sec.1	The section has been restructured. Text regarding follow up of manufacturing by third party surveyor has been removed. A number of terms, symbols, Greek characters and abbreviations have been added.
Guidance note on tower revised	[1.1]	The second paragraph in the guidance note has been revised.
Update of DNV GL reference documents	[1.5], whole document	New DNV GL standards and recommended practices have replaced DNV documents.
New floater and degree of freedom (DOF) figures included	[1.4]	The text describing typical floaters and boundary conditions has been improved to clarify the six degrees of freedom of a floating turbine, and two new figures have been included replacing the previous in DNV-OS-J103.
Change from safety class to consequence class	[2.2.1]	Low safety class has been removed and the other two safety classes have been replaced by consequence classes, adopting definitions from DNVGL-OS-E301, to avoid confusion with the safety class of the wind turbine.
Definition of accidental limit state (ALS) revised	[2.4.1.2]	The definition of ALS has been revised; presence of nonlinear environmental conditions has been replaced by presence of abnormal environmental conditions.

Topic	*Reference*	*Description*
Requirements on model test and software validation revised	[2.6.2.1], [4.6.2.3]	Requirements for model tests have been revised to only require model tests for novel designs. A subsection [4.6.2.3] regarding validation of software has been added.
Environmental classes	[3.1]	The subsection has been restructured to clarify the environmental classes. The text has been revised and additional items added in subsection [3.1.2.5].
Wind gust revised	[3.2.2]	The guidance note on gust model has been expanded by an expression for reference wind speed that characterizes the intensity of the gust. In additon, a guidance note has been added in [3.2.2.7] and [3.2.2.11], and subsection [3.2.2.10] has been revised.
JONSWAP spectrum	[3.2.3.7]	An alternative regarding combination of JONSWAP spectrums has been added.
Deep water specification	[3.2.4]	A specification related to deep water has been removed.
Expanded seismicity text	[3.4]	The text about seismicity has been expanded, and reference to DNVGL-ST-0437 and ISO 19901-2 introduced.
Return period of accidental load modified and added paragraph regarding current	[3.6]	The title of the subsection has been shortened. References have been updated The return period for accidental loads the in guidance note in subsection [3.6.1.3] has been changed from 1000 to 500 years, see also changes in section [4.7.1.1] and [8.3.3.1] The subsection [3.6.1.10] on current has been added. The return period for accidental loads in the guidance note in [3.6.1.3] has been changed from 1000 to 500 years, see also changes in [4.7.1.1] and [8.3.3.1]. [3.6.1.10] regarding current has been added.
Requirements for marine growth added	[3.7.3]	A requirement related to assessment of site specific conditions and clarification regarding structural components have been added.

Topic	*Reference*	*Description*
Storm conditions added	[4.1.2.1]	Highlighted that storm conditions with high waves can cause governing extreme loads.
Coupling effect added	[4.2.1.4]	Highlighted that coupling effects may be important.
Load categories added	[4.3.1.2]	Load categories added and restructuring of list.
Environmental loads	[4.3.2]	The values for load category environmental loads in ALS in Table 4-1 and Table 4-2 have been revised to abnormal. A load category for prestressing (P) has been added.
Ship impact loads	[4.5.2]	The text about ship impact loads has been expanded by text carried over from the replaced DNV-OS-J101 The text about ship impact loads has been expanded.
Survival condition	[4.6.2.1]	The paragraph stating that the survival condition needs shorter survival time has been removed.
Floater specific design load cases (DLC) included	[4.6.3]	Floater-specific DLCs have been included. A requirement for using the applicable current model in the DLCs given in DNVGL-ST-0437 has been added.
Return period for accidental loads revised	[4.7.1.1], [3.6.1.3], [8.3.3.1]	The return period for accidental loads has been reduced to 500 years.
Additional guidance for variance of draught	[4.9.3.1]	Guidance added regarding effects influencing the operational draught.
Causes for wave slamming added	[4.9.6.1]	A number of additional effects that may cause wave slamming have been added.
Limit states clarified and load category P added	[5.1.1.1]	Example of ALS for intact and damage structure has been clarified. Description of the ultimate limit state (ULS) for abnormal loads has been revised and reference to DNVGL-ST-0437 added. Two load factors are given (differentiating between consequence class 1 and 2). For ULS load factor set (b), load factors for the situation where there is a risk for excessive dynamic excitations have been added. A load category P has been added, applicable for concrete structures.

Topic	*Reference*	*Description*
Requirement related to structural cross joints moved	[6.1.1.2], [6.2.2.3]	A requirement related to cross joints and plate material has been moved to [6.2.2.3].
Guidance note on ASTM bolts deleted	[6.2.3]	A guidance note related to the ASTM standard for bolts has been deleted.
Structural categorization added	[6.2.4]	New subsection on structural categorization has been included.
Guidance note on design of fibre ropes deleted	[6.6.4.1]	A guidance note related to design of fibre ropes has been deleted as the information is available in DNVGL-RP-E304.
Guidance note on decommissioning added	[6.8]	A guidance note related to decommissioning and methods for removal has been added.
Requirement related to concrete structures moved	[7.1.1.4], [7.5]	Requirements related to concrete structures have been moved from [7.1.8] to a new subsection for concrete structures, [7.5].
Guidance note on optimum floater design deleted	[7.1.2.2]	Guidance note related to optimum floater design has been deleted as it did not give useful information.
Clarification related to Mathieu instability (MI) and vortex induced motions (VIM)	[7.1.3.1]	MI and VIM shall be considered in design.
Guidance note revised regarding weather criterion for installation of components	[7.1.4]	Guidance note on the use of same weather criterion for installation of different components in a wind farm.
New subsections on scantlings	[7.1.6], [7.1.7]	New subsections on scantlings have been included.
New SN-curve table	[7.3.1.4]	Table 7-4 with criteria for selection of SN-curves has been included.
Revised design fatigue factors (DFFs)	Table 7-5	Requirements for design fatigue factors in Table 7-5 have been replaced by new requirements which are in compliance with requirements for fatigue design of bottom-fixed support structures for wind turbines in DNVGL-ST-0126. In the footnote it has been clarified that inspections are by non-destructive testing (NDT).
Clarification on corrosion protection	[7.3.1.5]	Applicability of corrosion protection strategies have been clarified.

Topic	Reference	Description
New table with DFFs for steel tendons	Table 7-7	Requirements for DFFs for fatigue design of steel tendons have been changed by referring to Table 7-5 and formerly Table 7-2 have been replaced by Table 7-7 with tendon-specific DFF requirements.
Subsections on special floater provisions moved	[7.6]	The subsections with special provisions for semi-submersibles, tension leg platforms (TLPs) and spars have been moved to the end of the Sec.7.
Guidance note added	[8.2.2.2]	Guidance note related to governing ULS cases has been included.
Combined spectrum approach deleted	[8.2.5.3]	Guidance related to use of combined spectrum approach has been deleted.
Out-of-plane bending included	[8.2.5.4]	A new subsection regarding out-of-plane bending of in-chain links has been included.
New table with DFFs for mooring chains	[8.2.5.1]	Requirements for DFF for fatigue design of mooring chains by reference to general DFF requirements for steel in Sec.7 have been replaced by Table 8-2 with chain-specific DFF requirements.
Tendon slack	[8.3.3.1]	Guidance regarding tendon slack has been revised, and design against slack in the ALS has been allowed as an alternative to designing against slack in the ULS. The return period for ALS in damage condition is reduce to 500 years.
Guidance note included	[9.1.3.2]	A guidance note related to typical confidence levels have been included.
Shared anchors	[9.1.6]	New subsection on shared anchors has been included.
Scour	[9.1.7]	New subsection on scour and scour prevention has been included.
Prestressed rock anchors	[9.8]	Explanatory text about prestressed rock anchors has been added for clarification. Terms have been revisited and nomenclature updated for clarification and alignment with common terminology for prestressed rock anchors.
Terminology of floaters revised	Sec.10, whole document	Column-stablized revised to semi-submersible, ship-shaped revised to barge, deep-draught revised to spar.

Topic	*Reference*	*Description*
Stability requirements updated	[10.1]	Requirements for stability in the case of manned structures have been removed, because it is explicitly stated that the standard is based on the prerequisite that the floating wind turbine structures are unmanned.
Requirement related to manufacturing added	[10.1.1.10]	A clarification related to performance of a lightweight survey or inclination test when a unit is manufactured on more than one yard has been included.
Clarification related to calculation of wind loads	[10.1.1.11]	Clarification related to vertical axis turbine has been added in the guidance note.
Emergency response	[10.1.1.12]	New subsection is included with the requirement that an emergency response plan shall be worked out.
Position of the rotor plane	[10.2.1.3]	Text adjusted from parallel to perpendicular rotor plane.
Wind speeds and heeling	[10.2.1.5], [10.3.1.5]	A requirement stating that wind speeds shall be superimposed from any direction has been included.
Damage stability	[10.3]	In [10.3.1.1] an advice to perform nonlinear collision analysis or to meet the damage stability requirements have been added. New subsection [10.3.1.2] describing collision analysis has been included. The guidance note in [10.3.1.3] has been updated with additional text about flooding. In addition, it has been added that also environmental impact and salvage cost should be included in a cost-benefit analysis. The text in [10.3.1.4] has been adjusted. New subsection [10.3.1.5] has been included. In [10.3.6] some adjustments and clarifications have been made regarding assumptions for extent of damage. Specific requirements for spars and TLPs have been added.
Inclination angle	[10.3.4.2]	A third bullet point regarding inclination angle has been added.
Requirements for spar and TLP related to extent of damage included	[10.3.6.4], [10.3.6.5]	Specific requirements for spar and TLP related to extent of damage have been included.

Topic	*Reference*	*Description*
Control system restructured and rewritten	Sec.11	The section on motion control system has been restructured and partly rewritten.
Power generation systems	[12.2.9]	Requirement for power supply for winches and other equipment has been revised.
Corrosion protection revised	Sec.13	The splash zone definitions in [13.1.2] have been modified by removing components referring to floater motions (external zone) and sloshing (internal zone), since - without a statement of return periods - these components were incompletely defined. The thus modified splash zone definitions imply splash zone heights which are still considered adequate. The requirements for corrosion allowance in [13.1.3] have been rephrased, the 80% threshold on relative humidity have been removed. In [13.1.3.2] a guidance note has been added below Table 13-1. Furthermore, the requirement for chain replacement when the chain diameter is reduced by 2% in Table 13-1 has been removed as being out of place in a table of only recommendations for minimum corrosion allowance. In [13.1.4], the requirement to consider altered corrosion rates associated with energized cables at cable attachment points has been changed to a recommendation, as there is no simple method available to assess the induced electrical field and its effects on the corrosion. The item is kept as it is important to raise the awareness regarding this issue and a guidance note is included with explanation.
Section on in-service inspection, maintenance and monitoring updated	Sec.15	The section has been expanded by including anchors, mooring chain and steel tendons. A requirement for measurement of plate thicknesses by ultrasonic testing, applicable to corroded structures, has been included.

Topic	*Reference*	*Description*
Power cables revised	Sec.16	The section on power cable design has been restructured and partly rewritten. Utilization factor requirements in Table 16-2 have been revised after a recalibration has been carried out to satisfy partial safety factor requirements in consequence class 1. Ancillary(ies) changed to accessory(ies)

Editorial corrections

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

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SECTION 1 GENERAL

1.1 Introduction

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

The standard covers structural design of floating wind turbine structures. The standard gives provisions for the floater motion control system and the control system for the wind turbine – whether these systems are separate or combined – to the extent necessary in the context of structural design. The standard also gives provisions for transportation, installation and inspection to the extent necessary in the context of structural design. The design principles and overall technical requirements are specified in the standard. Wherever possible, the standard makes reference to requirements set forth in DNVGL-ST-0126.

The standard shall be used for design of support structures and station keeping systems for floating wind turbines.

The standard does not cover design of wind turbine components such as nacelle, rotor, generator and gearbox. For structural design of rotor blades DNVGL-ST-0376 applies. For structural design of wind turbine components for which no DNV GL standard exists, the IEC 61400-1 standard applies.

The tower, which usually extends from a defined elevation 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. The actual stiffness and mass distribution of the floating system shall be considered in the design of both the tower and the substructure.

Guidance note:

For a type-approved tower, the stiffness of the tower forms part of the basis for the approval. It is advisable, if possible, not to change the mass and stiffness distributions over the height of the tower relative to those assumed for the type approval. However, for floating structures it may be impractical to maintain the tower stiffness, since the turbine type approval which the tower approval is part of will generally assume the tower is connected to a rigid foundation or a foundation stiffer than the floating structure.

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The standard is 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 floating wind turbine structure.

The standard is in principle written for site-specific design; however, it may be suitable with a view to an expectation of mass production to design a floating wind turbine structure not for a specific site but rather for a class of environmental conditions and then, for each application, qualify the structure for the specific location in accordance with this standard.

Guidance note:

A class of environmental conditions, defined to be used as a target for design of floating units for mass production, would have to cover environmental conditions in a broad sense, i.e. including a range of water depths and ultimately also a range of wind turbines, since the turbine is expected to influence the response of the floating support structure.

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1.2 Objective

The standard specifies general principles and requirements for the structural design of floating wind turbine structures.

The objectives of this standard are to:

— provide an internationally acceptable level of safety by defining minimum requirements for structures and structural components (in combination with referenced standards, recommended practices, guidelines, etc.)

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

1.3 Scope

This standard gives requirements for the following:

— design principles
— selection of material and extent of inspection in manufacturing yard
— design loads
— load effect analyses
— load combinations
— structural design
— station keeping
— anchoring
— corrosion protection
— transport and installation
— in-service inspection
— power cable design.

This standard specifies requirements for the design of floating 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 floating 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. A basic premise for such analyses should be that all cost effective risk control options have been implemented. Technology qualification procedures may be helpful in this context.

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

Guidance note:

Risk acceptance criteria may be taken in accordance with IMO MSC/Circ.1023−MEPC/Circ.392 *Guidelines for Formal Safety Assessment*.

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The specific requirements of this standard reflect what was deemed cost effective means of managing the risks associated with floating wind turbine structures at the time of issue of this standard. Technology developments after that point in time may provide new means of cost effective risk reduction. Should relevant cost benefit assessment show that use of such new technology would provide cost effective risk reduction, such new technology should be implemented on new floating structures where it fits.

1.4 Application

1.4.1 General

This standard is applicable to all types of support structures and station keeping systems for floating wind turbines.

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

The standard is 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 floating wind turbine structure.

The standard is in principle written for site-specific design; however, it may be suitable with a view to an expectation of mass production to design a floating wind turbine structure not for a specific site but rather for a class of environmental conditions and then, for each application, qualify the structure for the specific location in accordance with this standard.

Guidance note:

A class of environmental conditions, defined to be used as a target for design of floating units for mass production, would have to cover environmental conditions in a broad sense, i.e. including a range of water depths and ultimately also a range of wind turbines, since the turbine is expected to influence the response of the floating support structure.

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1.4.2 Typical floaters and boundary conditions

1.4.2.1 Support structures for floating wind turbines may either be compliant, or restrained for some of the global modes of motions, also referred to as degrees of freedom (DOF): surge, sway, heave, roll, pitch and yaw. For easy reference, Table 1-1 shows different floater types with basis in floating offshore structures. This overview may not cover all possible solutions. However, the main types are captured. Restrained modes will not imply a total fixation, but displacements in the order of centimetres will occur (e.g. an elastic stretch of a tension leg platform (TLP) tendon) compared to displacements in the order of metres for a compliant mode. In Table 1-1 . Four of the most common floater types are shown in Figure 1-1.

Table 1-1 Typical floaters and boundary conditions

Type	*Surge*	*Sway*	*Heave*	*Roll*	*Pitch*	*Yaw*
Deep draught floaters (DDF)[1)]	C[6)]	C	C	C	C	C
Spar	C	C	C	C	C	C
Column-stabilized	C	C	C	C	C	C
Semi-submersibles [5)]	C	C	C	C	C	C
Barges	C	C	C	C	C	C
Tension leg platforms (TLP)	C	C	R	R	R	C
Heave-restrained TLP (HRTLP) [2)]	C	C	R	C	C	C
Heave-restrained DDF (HRDDF)[3)]	C	C	R	C	C	C
Ship-shaped	C	C	C	C	C	C
Articulated tower[4)]	C	C	R	C	C	C
Compliant tower[4)]	C	C	R	R	R	R

1) Classic, truss & cell spar, deep draught semi, buoys.
2) Special type TLP which has not been built, but proposed and developed to a certain level.
3) Special type DDF.
4) These structures are fixed to the seabed as fixed structures, but use buoyancy as a vital part of the loadbearing system.
5) Semi-submersibles are usually column-stabilized units.
6) C denotes compliant and R denotes restrained.

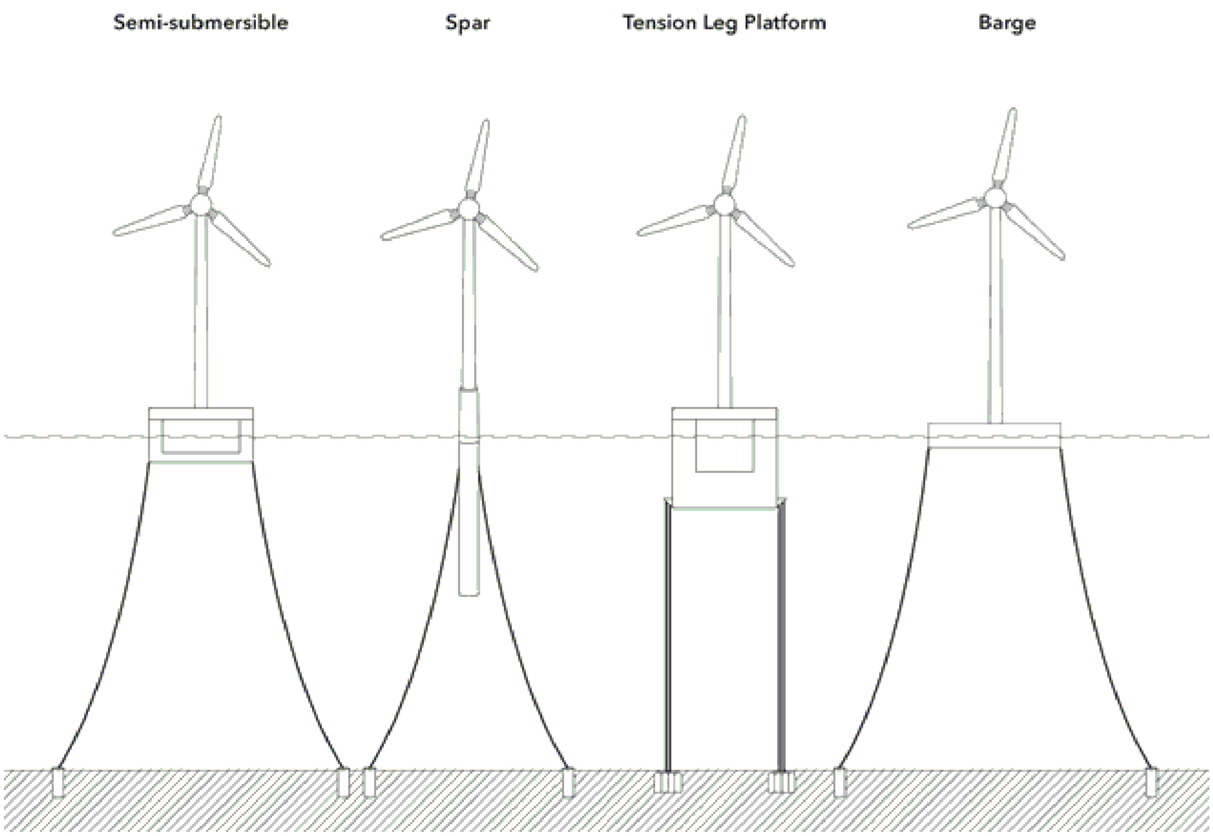
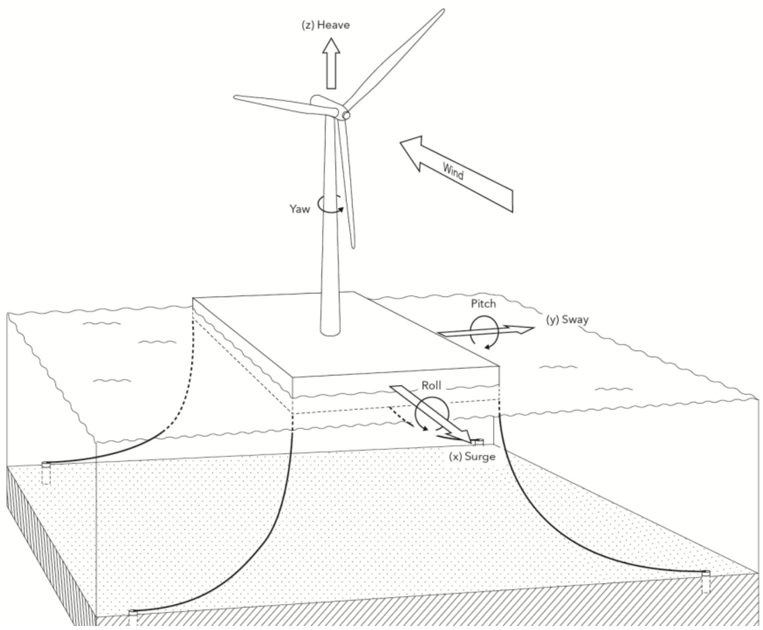


Figure 1-1 Four common floater types for wind turbines

1.4.2.2 For the floaters listed in Table 1-1, the six referenced degrees of freedom relate to the floater, i.e. the surge and sway axes follow the orientation of the floater. For example, for a ship-shaped floater, the surge axis is the longitudinal axis. For a spar, which is axisymmetric, the surge and sway axes of the floater are not predefined by the spar structure itself, but their orientation may be dictated by the station keeping system as this is usually not axisymmetric.

1.4.2.3 In some contexts, it is convenient to refer the six degrees of freedom to the orientation of the wind turbine rather than to the orientation of the floater. This is the case, for example, when reference is made to the control system of the wind turbine as this refers to the orientation of the turbine. This gives two coordinate systems, one for the floater and one for the turbine, and they are not necessarily the same. In particular, the coordinate system referring to the turbine will yaw with the turbine when the direction of the wind changes, while the coordinate system of the floater will stay put.

1.4.2.4 Figure 1-2 shows a floating wind turbine with the six degrees of freedom marked out. As indicated in the legend in Figure 1-2, the surge, sway and yaw axes follow the orientation of the floater.



Surge	Displacement along the longitudinal axis
Sway	Displacement along the lateral axis
Heave	Displacement along the vertical axis
Roll	Rotation about the longitudinal axis
Pitch	Rotation about the lateral axis
Yaw	Rotation about the vertical axis

Figure 1-2 Degree of freedom of a floating wind turbine

1.5 References

The DNV GL documents listed in Table 1-2 are referred to in this standard. The latest valid revision of each of the DNV GL reference documents in Table 1-2 applies.

Table 1-2 DNV GL documents

Document code	*Title*
DNV-RP-C201	Buckling Strength of Plated Structures
DNVGL-CG-0129	Fatigue assessment of ship structures
DNVGL-OS-A101	Safety principles and arrangements
DNVGL-OS-B101	Metallic materials
DNVGL-OS-C101	Design of offshore steel structures, general – LRFD method
DNVGL-OS-C103	Structural design of column stabilised units – LRFD method
DNVGL-OS-C105	Structural design of TLPs – LRFD method
DNVGL-OS-C106	Structural design of deep draught floating units – LRFD method
DNVGL-OS-C301	Stability and watertight integrity
DNVGL-OS-C401	Fabrication and testing of offshore structures
DNVGL-OS-D101	Marine and machinery systems and equipment

Document code	*Title*
DNVGL-OS-D201	Electrical installations
DNVGL-OS-D202	Automation, safety, and telecommunication systems
DNVGL-OS-E301	Position mooring
DNVGL-OS-E302	Offshore mooring chain
DNVGL-OS-E303	Offshore fibre ropes
DNVGL-OS-E304	Offshore mooring steel wire ropes
DNVGL-OS-F201	Dynamic risers
DNVGL-OTG-13	Prediction of air gap for column-stabilized units
DNVGL-OTG-14	Horizontal wave impact loads for column-stabilized units
DNVGL-RP-0360	Subsea power cables in shallow water
DNVGL-RP-0416	Corrosion protection for wind turbines
DNVGL-RP-A203	Technology qualification
DNVGL-RP-C103	Column-stabilised units
DNVGL-RP-C104	Self-elevating units
DNVGL-RP-C202	Buckling strength of shells
DNVGL-RP-C203	Fatigue design of offshore steel structures
DNVGL-RP-C205	Environmental conditions and environmental loads
DNVGL-RP-C207	Statistical representation of soil data
DNVGL-RP-C208	Determination of structural capacity by non-linear finite element analysis
DNVGL-RP-C212	Offshore soil mechanics and geotechnical engineering
DNVGL-RP-E301	Design and installation of fluke anchors
DNVGL-RP-E302	Design and installation of plate anchors in clay
DNVGL-RP-E303	Geotechnical design and installation of suction anchors in clay
DNVGL-RP-E304	Damage assessment of fibre ropes for offshore mooring
DNVGL-RP-E305	Design, testing and analysis of offshore fibre ropes
DNVGL-RP-F105	Free spanning pipelines
DNVGL-RP-F107	Risk assessment of pipeline protection
DNVGL-RP-F109	On-bottom stability design of submarine pipelines
DNVGL-RP-F203	Riser interference
DNVGL-RP-F204	Riser fatigue
DNVGL-RP-F205	Global performance analysis of deepwater floating structures
DNVGL-RP-F401	Electrical power cables in subsea applications
DNVGL-RP-N101	Risk management in marine and subsea operations

Document code	*Title*
DNVGL-RP-N103	Modelling and analysis of marine operations
DNVGL-RU-HSLC	DNV GL rules for classification: High speed and light craft
DNVGL-RU-OU-0102	Floating production, storage and loading units
DNVGL-SE-0073	Project certification of wind farms according to IEC 61400-22
DNVGL-SE-0074	Type and component certification of wind turbines according to IEC 61400-22
DNVGL-SE-0176	Certification of navigation and aviation aids of offshore wind farms
DNVGL-SE-0190	Project certification of wind power plants
DNVGL-SE-0422	Certification of floating wind turbines (planned published 2018)
DNVGL-ST-0054	Transport and installation of wind power plants
DNVGL-ST-0076	Design of electrical installations for wind turbines
DNVGL-ST-0126	Support structures for wind turbines
DNVGL-ST-0359	Subsea power cables for wind power plants
DNVGL-ST-0361	Machinery for wind turbines
DNVGL-ST-0376	Rotor blades for wind turbines
DNVGL-ST-0377	Standard for shipboard lifting appliances
DNVGL-ST-0378	Standard for offshore and platform lifting appliances
DNVGL-ST-0437	Loads and site conditions for wind turbines
DNVGL-ST-0438	Control and protection systems for wind turbines
DNVGL-ST-C501	Composite components
DNVGL-ST-C502	Offshore concrete structures
DNVGL-ST-N001	Marine operations and marine warranty

The documents in Table 1-3 include requirements, acceptable methods and useful guidance to supplement the contents of this standard. The current revision of each of the reference documents in Table 1-3, valid at the time of publishing this standard, applies.

Table 1-3 External documents

Reference	*Title*
ANSI/ASME B16.5	Pipe Flanges and Flanged Fittings
API Spec. 16A	Specification for Drill Through Equipment
API Spec. 17J	Specification for Unbonded Flexible Pipe
API Spec. 17L1	Specification of Flexible Pipe Ancillary Equipment
API RP 2T	Planning, Designing and Constructing Tension Leg Platforms
BS 7910	Guide on methods for assessing the acceptability of flaws in fusion welded structures

Reference	Title
EEMUA 194	Guidelines for materials selection and corrosion control for subsea oil and gas production equipment
EN 1537	Execution of special geotechnical work – Ground Anchors
EN 1993-1-1	Eurocode 3: Design of steel structures – Part 1: General rules and rules for buildings
EN 1993-1-6	Eurocode 3: Design of steel structures – Part 6: Strength and stability of shell structures
EN 1993-1-8	Eurocode 3: Design of steel structures – Part 8: Design of joints
EN 1997-1	Eurocode 7: Geotechnical Design – Part 1: General rules
EN ISO 22477-5	
IEC 61400-1	Wind Turbines – Part 1: Design requirements
IEC 61400-3	Wind Turbines – Part 3: Design requirements for offshore wind turbines
IEC 61892-6	Mobile and fixed offshore units – Electrical installations – Part 6: Installation
IMO MSC/ Circ.1023−MEPC/ Circ.392	Guidelines for Formal Safety Assessment
ISO 898-1	Mechanical properties of fasteners made of carbon steel and alloy steel – Part 1: Bolts, screws and studs with specified property classes – Coarse thread and fine pinch thread
ISO 13628-5	Petroleum and natural gas industries – Design and operation of subsea production systems – Part 5: Subsea umbilicals
ISO 19901-2	Petroleum and natural gas industries – Specific requirements for offshore structures – Part 2: Seismic design procedures and criteria
NORSOK N-004	Design of steel structures
NORSOK N-006	Assessment of structural integrity for existing offshore load-bearing structures
PTI DC35.1	Recommendations for Prestressed Rock and Soil Anchors

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

The provision for using non-DNV GL codes or standards is that the same safety level as the one resulting for designs in accordance with this standard is obtained.

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

When code checks are performed in accordance with other codes than DNV GL documents, the resistance and material factors as given in the respective codes shall be used.

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

1.6 Definitions and abbreviations

1.6.1 Definition of verbal forms

Table 1-4 Definition of verbal forms

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

1.6.2 Definition of terms

Table 1-5 Definition of terms

Term	*Explanation*
abnormal environmental loads	loads with annual probability of exceedance of 10^{-3} i.e. loads with return period 500 years
abnormal wind load	wind load resulting from one of a number of severe fault situations for the wind turbine, which result in activation of system protection functions Abnormal wind loads are in general less likely to occur than loads from any of the normal wind load cases considered for the ULS.
accidental limit states (ALS)	for a floating wind turbine unit, accidental limit states are survival conditions in a damaged condition or in the presence of strongly nonlinear environmental conditions
anchor	structural device to transfer forces from mooring lines or tendons to seabed soils or rock For anchors attached to mooring lines, the anchor is rationalized as the anchor itself plus the embedded part of the mooring line.
articulated tower	tower which is flexibly connected to the seabed through a cardan joint and held vertically by the buoyancy force acting on it The structure is free to oscillate in any direction and does not transfer any bending moment to its foundation.
barge	free-surface stabilized structure A barge has large water plane area and relatively small draught.
bell-mouth	tapered section at the end of a pipe, often at the outlet of an I-tube or a J-tube
bilge system	system for pumping and removing bilge water
bonded length	length of rock anchor bonded to grout/rock and transferring anchor loads to the surrounding rock mass
chinese fingers	cable grip (stocking), made up of braided wire rope, used to pull or support cable tension

Term	*Explanation*
co-directional	term used in the context of wind and waves to indicate that the wind and the waves act in the same direction Another term is collinear.
collision ring	inner bulkhead in the splash zone with the purpose of providing a second barrier in case of damage or rupture to external hull skin
column-stabilized	structure dependent of buoyancy of widely spaced columns for floatation and stability
compliant tower	flexible structure with a controlled mass and stiffness characteristics so as to mitigate the effects of wind, wave and current forces Natural periods are usually greater than 25 sec so compliant towers are generally well outside wave periods.
connector arrangement	assembly of structural components, one of which is pre-installed on the floater structure and one of which is assembled to the cable end during installation, designed to transfer cable loads to the floater structure throughout the specified service life of the power cable system
deep draught floater (DDF)	spar or similar type platform with a relatively large draught compared to barges and semi-submersibles
deep water	deep waters are characterized by wavelengths shorter than about twice the water depth
design life	the period of time over which the floating support structure and its station keeping system is designed to provide an acceptable minimum level of safety
downflooding	any flooding of the interior of any part of the buoyant structure of a floating unit through openings which cannot be closed watertight or weathertight, as appropriate, in order to meet the intact or damage stability criteria, or which are required for operational reasons to be left open
downflooding angle	the angle of inclination of a floater at which downflooding will begin to take place through openings which are unprotected against flooding, i.e. openings that cannot be closed water- or weathertight
fatigue limit states (FLS)	fatigue crack growth and associated failure of structural details due to stress concentration and damage accumulation under the action of repeated loading
floating wind turbine unit	term used for the entire system consisting of wind turbine, floating support structure and station keeping system A system of two or more wind turbines mounted on the same floater is rationalized as one floating wind turbine unit. The term is also used in cases, where the support structure is of a kind that the floating wind turbine is only partly supported by buoyancy, for example when the support structure consists of an articulated tower or a compliant tower.
heel	a tilt, as of a boat, to one side Heel is used here in the context of wind heeling when floating stability is the issue.
high frequency	frequency band relating to fast-varying responses with frequencies above the typical wave frequency range Examples include ringing and springing responses in TLPs.
hub height	height of centre of swept area of wind turbine rotor, measured from mean water level (MWL), see DNVGL-ST-0126
lightweight	the invariable weight of the floating structure, i.e. the basis for calculating the loading conditions for evaluation of floating stability Anchors, mooring lines and cables are to be excluded from the lightweight. Weights of mooring lines and cables are to be included in the loading conditions as variable loads.

Term	*Explanation*
lock-off tension	the prestressing load transferred into a rock anchor immediately following proof-loading of the anchor The lock-off tension is the permanent tensile lock-off load that a prestressed rock anchor will be subject to before hook-up of any tendon or mooring line.
low frequency	frequency band relating to slowly varying responses with frequencies below the typical wave frequency range Examples of such slowly varying responses include slowly varying surge and sway motions of column-stabilized and ship-shaped units and slowly varying roll and pitch motions of deep draught floaters.
Mathieu instability (MI)	type of instability which is caused by coupling between the heave and pitch motions MI typically occurs as the result of large heave motion if the natural period of heave motion comes close to or equals half the natural period of pitch, causing the pitch motion to increase significantly.
mean water level	long-term average water level, i.e. the arithmetic mean of all sea levels measured over a long period
misalignment	wind and wave misalignment is a term which designates that the wind and the waves at a given point in time and space are not co-directional, i.e. they act or propagate in different directions Misalignment is also used as a term for the deviation between the mean wind direction and the rotor axis.
meandering wakes	the unsteady behavior of the wind turbine wake that is characterized by a wake following the large-scale turbulence structures.
mooring line	strong slender line, such as chain, rope or wire, in catenary and taut mooring systems of floating units Note that tendons used for tension leg platforms are not referred to as mooring lines.
mooring system	system made up of mooring lines, anchors and connectors, used for station keeping of a floating structure
operational ballast water level	ballast water level in a floater in the permanent operational condition on site Note that the ballast water level that is intended to be used for inspection and repair might be a ballast water level outside the range of operational ballast water levels.
operational draught	draught of floater in the permanent operational condition on site Note that the draught that is intended to be used for inspection and repair might be a draught outside the range of operational draughts.
P-delta effect	changes in overturning moment, shear force and/or the axial force distribution of a structure or structural component when it is subject to a lateral displacement, such as that caused by a tilt The P-delta effect is a destabilizing moment equal to the force of gravity multiplied by the horizontal displacement that a structure undergoes as a result of a lateral displacement.
power cable components	all components that constitute the cable cross-section, such as conductors, insulation, tapes, armour wires, and sheath
power cable system	complete power cable with terminations and permanent components, such as connector at floater interface, bend stiffener, buoyancy modules, and bend restrictors

Term	Explanation
redundancy	ability of a component or system to maintain or restore its function after a failure of a member or connection has occurred Redundancy may be achieved for instance by strengthening or introducing alternative load paths. For example, if one mooring line in a mooring system is lost and the remaining part of the mooring system meets the ALS criterion, which is survival for at least a one-year load, then the initial undamaged mooring system is said to be redundant.
return period	average time between two events characterized by a given magnitude Measured in number of years, the return period is the inverse of the annual probability of exceedance of an event such as the occurrence of a wave height, i.e. a 50-year wave height has a 2% probability of being exceeded in any one year.
ringing	high-frequency transient resonant structural response due to higher-order wave loads
rotor-nacelle assembly	part of wind turbine carried by the support structure
scour zone	external region of a structure which is located at the seabed and which is exposed to scour
semi-submersible	buoyancy and free-surface stabilized structure with a relatively shallow draught A number of large columns are linked to each other by bracings. The columns provide the ballast and flotation stability (column-stabilized)
serviceability limit states (SLS)	disruption of normal operations due to deterioration or loss of routine functionality, e.g. exceedance of normal operation criteria The SLS imply deformations in excess of tolerance without exceeding the load-carrying capacity, i.e., they correspond to tolerance criteria applicable to normal use and durability. Unacceptable deformations and excessive vibrations are typical examples of the SLS.
set-down	kinematic coupling between the horizontal surge/sway motions and the vertical heave motions For a TLP, set-down refers to the vertical downward movement of the hull when the platform moves in its compliant modes (surge, sway and yaw).
shallow water	shallow waters are characterized by wavelengths greater than 10 times the water depth
spar	weight-buoyancy stabilized structure with a relatively large draught compared to barges, semi-submersibles and TLPs A spar can consist of multi-vertical columns or a single column with or without moonpool (e.g. classic, truss and cell spar).
specified minimum yield strength (SMYS)	minimum yield strength prescribed by the specification or standard under which the material is purchased
specified minimum tensile strength (SMTS)	minimum tensile strength prescribed by the specification or standard under which the material is purchased
splash zone	external or internal surfaces of a structure which are intermittently wetted by tide or waves or both
springing	high-frequency stationary resonant structural response due to sum-frequency wave loads
station keeping system	system to maintain a floating structure in a fixed position relative to a fixed point or within a defined sector relative to the fixed point The station keeping system includes the mooring lines or tendons, as applicable, as well as the anchor foundations that transfer forces from the system to the seabed.
still water level	average water surface elevation at any instant, excluding local variations due to waves, but including the effects of tides and storm surges

Term	*Explanation*
suitability test	test to define the creep and load-loss characteristics of prestressed rock anchors at proof and lock-off loads
support structure	structure below the yaw system of the rotor-nacelle assembly and includes tower structure, substructure and station keeping system
tendon	structural component used as part of the station keeping system for a TLP, also referred to as tether
tension leg platform (TLP)	vertically moored floating structure whose station keeping system consists of tethers or tendons anchored at the seabed
TLP tendon system	comprises all components between, and including the top connection(s) to the hull and the bottom connection(s) to the foundation(s) Guidelines, control lines, umbilicals etc. for tendon service and or other permanent installation aids are considered to be included as part of the tendon system
tower	structural component, which forms a part of the support structure for a wind turbine, usually extending from somewhere above the still water level to just below the nacelle of the wind turbine
ultimate limit states (ULS)	failure or collapse of all or part of a structure due to loss of structural stiffness or exceedance of load-carrying capacity Overturning, capsizing, yielding and buckling are typical examples of the ULS.
unbonded length	initial length of rock anchor which is free to move relative to the surrounding grout/rock
wave frequency	frequency band applicable to responses of offshore structures located in the wave active zone, corresponding to the frequency range of the incoming waves, typically responses with periods between 4 s and 25 s
wave frequency motion	motion induced by first-order wave loads in the frequency range of the incoming waves
wind heeling moment	product of wind force and wind force arm which produces the heel in question
wind turbine	system which converts kinetic energy in the wind to electrical energy In this standard the term is used to designate the rotor-nacelle assembly.
wind turbine structure	term denoting the support structure of a wind turbine, i.e. the tower, the substructure, the foundation for bottom-fixed substructures and the station keeping system for floating substructures, but excluding the wind turbine (the rotor-nacelle assembly) itself

1.6.3 Symbols and equations

Table 1-6 Latin characters

Symbol	*Definition*
a	coefficient
a_v	vertical acceleration
b	coefficient
b	breadth of load area
b_e	effective flange width

Symbol	Definition
c	wave celerity
e	void ratio
f	frequency
f_y	minimum yield stress
f_{yd}	design yield strength
g, g_0	acceleration of gravity
h_{op}	vertical distance from load point to position of maximum filling
k	wave number
k_a	correction factor for aspect ratio of plate field
k_m	bending moment factor
k_{pp}	fixation parameter for plate
k_{ps}	fixation parameter for stiffeners
k_r	correction factor for curvature perpendicular to the stiffeners
k_τ	shear force factor
l	stiffener span
l_0	distance between points of zero bending moment
n	number of samples
n_C	number of applied cycles
$n_{C,i}$	number of stress cycles
p	pressure
p_d	design pressure
p_{dyn}	dynamic pressure
p_e	dynamic (environmental) sea pressure
p_s	static sea pressure
p_{dyn}	dynamic pressure
r_c	radius of curvature
s	stiffener spacing
s_u	undrained shear strength of clay
t	time
t	thickness
t_0	element thickness
u	wind speed
x	distance of (wave) propagation

Symbol	*Definition*
z	height, vertical distance
z_b	vertical distance from moulded baseline to load point
A	righting moment for floating stability
A_{rotor}	swept rotor area
B	wind heeling moment
C_e	flange parameter
C_T	thrust coefficient
D	deformation load
D	standard deviation operator
D_C	characteristic cumulative damage
D_D	design cumulative damage
D_D	vertical distance from moulded baseline to wave crest
E	environmental load
E	expected value operator
F	load
F_B	characteristic breaking strength of mooring line
F_d	design load
$F_{d,wl}$	design load on windlass
F_k	characteristic load
G	permanent load
H	wave height
H_c	wave crest height
H_S	significant wave height
N	number of sea states
N_C	number of cycles to failure
$N_{C,i}$	number of cycles to failure at the stress range $\Delta\sigma_i$
N_p	number of supported stiffeners on the girder span
N_s	number of stiffeners between considered section and nearest support
Q	variable functional load
P_{pd}	average design point load from stiffeners between considered section and nearest support
R	rotor radius
R	resistance
R_c	characteristic resistance

Symbol	*Definition*
$R_{cy,c}$	characteristic post-installation effect due to cyclic loading
R_d	design resistance
$R_{drag,c}$	characteristic resistance gained by further drag and penetration of the anchor after installation
$R_{fric,c}$	characteristic line friction on the seabed
R_H	horizontal force on anchor
$R_{H,max}$	horizontal anchor resistance
$R_{i,c}$	characteristic installation resistance
R_k	characteristic resistance
$R_{set\text{-}up,c}$	characteristic post-installation resistance owing to thixotropy and consolidation
R_V	vertical force on anchor
$R_{V,max}$	vertical anchor resistance
S	girder span
S	load effect
S	power spectral density
S_c	characteristic load effect
S_C	characteristic capacity of mooring line
S_d	design load effect
S_k	characteristic load effect
S_{mbs}	minimum breaking strength
T	duration
T	wave period
$T_{c,mean}$	characteristic mean tension
$T_{c,dyn}$	characteristic dynamic tension
T_d	design tension
T_E	extreme operational draught from moulded baseline to assigned load waterline
T_P	peak period
T_R	return period
T_S	duration of sea state
T_Z	zero-upcrossing period
U_{10}	10-minute mean wind speed
V	wind speed
V_{gust} Z_g	reference wind speed for gust section modulus

Table 1-7 Greek characters

Symbol	*Definition*
α	scale parameter
α	angle
β	shape parameter
γ'	submerged unit weight of soil
γ_{c}	consequence factor
γ_{f}	load factor
$\gamma_{f,E}$	load factor for environmental loads
$\gamma_{f,G,}$	load factor for permanent loads
$\gamma_{f,G,Q}$	load factor for permanent loads and variables functional loads
γ_{m}	material factor
γ_{mean}	load factor, mean tension
γ_{dyn}	load factor, dynamic tension
Δ	fully loaded displacement
Δ_{σ}	stress range
η	utilization factor
θ	angle, phase angle
λ	wavelength
μ	mean value
ν	Poisson's ratio
ρ	density of liquid, density of seawater, density of air
σ	standard deviation
σ_{d}	design material strength
σ_{k}	characteristic material strength
σ_{jd}	equivalent design stress for global in-plane membrane stress
σ_{pd}	design bending stress
σ_{U}	standard deviation of wind speed
τ_{p}	stress (half of design yield strength)
φ	friction angle

Symbol	Definition
ω	angular wave frequency

1.6.4 Abbreviations

Table 1-8 Abbreviations

Abbreviation	Full term
3-T	tension, time and temperature
AC	alternating current
ALS	accidental limit state
ASD	allowable stress design
BL	baseline
C	compliant
CFD	computational fluid dynamics
COG	centre of gravity
COV	coefficient of variation
DAE	differential algebraic equation
DDF	deep draught floater
DFF	design fatigue factor
DLC	design load case
DOF	degree of freedom
EEMUA	Engineering Equipment and Materials Users' Association
ECM	extreme current model
EOG	extreme operating gust
ESS	extreme sea state
EWLR	extreme water level
EWM	extreme wind speed model
FEM	finite element method
FL	floater
FLS	fatigue limit state
FMECA	failure modes, effects and criticality analysis
FORM	first order reliability method
FSD	functional system design
GDW	generalized dynamic wake

Abbreviation	*Full term*
GM	metacentric height
GSI	geological strength index
HAT	highest astronomical tide
HAWT	horizontal axis wind turbine
HF	high frequency
HRDDF	heave restrained DDF
HRTLP	heave restrained TLP
IEC	International Electrotechnical Commission
IMO	International Maritime Organization
IPC	individual pitch controller
ISO	International Organization of Standardization
LAT	lowest astronomical tide
LF	low frequency
LRFD	load and resistance factor design
MI	Mathieu instability
MIC	microbiologically induced corrosion
MSL	mean sea level
MWL	mean water level
NA	not applicable
NCM	normal current model
NDT	non-destructive testing
NSS	normal sea state
NTM	normal turbulence model
OS	DNV GL offshore standard
PTI	Post-Tensioning Institute
QTF	quadratic transfer function
R	restrained
RMR	rock mass rating
RNA	rotor-nacelle assembly
ROV	remotely operated vehicle
RQD	rock quality designation
SKS	station keeping system
SLS	serviceability limit state

Abbreviation	*Full term*
SMTS	specified minimum tensile strength
SMYS	specified minimum yield strength
SWL	still water level
TLP	tension leg platform
ULS	ultimate limit state
VAWT	vertical axis wind turbine
VCG	vertical centre of gravity
VIM	vortex induced motions
VIV	vortex induced vibrations
WF	wave frequency
WSD	working stress design
WT	wind turbine

SECTION 2 SAFETY PHILOSOPHY AND DESIGN PRINCIPLES

2.1 Introduction

2.1.1 Objective

The purpose of this section is to present the safety philosophy and the corresponding design principles applied in this standard.

2.1.2 Application

This section applies to all floating wind turbine structures which are to be designed in accordance with this standard.

2.2 Safety philosophy

2.2.1 Consequence class methodology

2.2.1.1 In this standard, structural safety is ensured by use of a consequence class methodology. The structure to be designed is classified into a consequence class based on the failure consequences. The classification is normally determined by the purpose of the structure. For each consequence class, a target safety level can be defined in terms of an annual probability of failure, see [2.2.2].

2.2.1.2 Two consequence classes are defined:

— Consequence class 1, where failure is unlikely to lead to unacceptable consequences such as loss of life, collision with an adjacent structure, and environmental impacts.
— Consequence class 2, where failure may well lead to unacceptable consequences of these types.

For floating wind turbine structures, which are unmanned during severe environmental loading conditions, the consequences of failure are mainly of an economic nature.

Guidance note:

Loss of structure is regarded as acceptable in conjunction with consequence class 1, as long as it does not cause harm to other structures.

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

2.2.1.3 Unless otherwise specified, the floating structure and its station keeping system shall be designed to consequence class 1. This requirement reflects that the floating structure is unmanned during severe environmental loading conditions.

2.2.1.4 For station keeping systems without redundancy, the design of the various components of the station keeping system shall be carried out to consequence class 2.

Guidance note:

The issue of redundancy of the station keeping system is dealt with in [8.1.1].

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

2.2.1.5 Design of floating wind turbine structures in accordance with this standard will lead to designs meeting the requirements for design to the normal safety class defined in DNVGL-ST-0126.

2.2.1.6 The different consequence classes applicable for different parts of the floating units and their station keeping systems are reflected in terms of different requirements for load factors. The requirements for material factors usually remain unchanged regardless of which consequence class is applicable for a particular wind farm or structure in question, see Sec.5.

In case the floating wind turbine is intended to be manned, a higher consequence class shall be used for the primary structure. In this case, a risk assessment shall be performed.

2.2.2 Target safety

2.2.2.1 The target safety level for structural design of floating wind turbine structures and their station keeping systems is a nominal annual probability of failure of 10^{-4} in consequence class 1 and 10^{-5} in consequence class 2. These target safety levels are aimed at for structures whose failures are ductile and have some reserve capacity. These target safety levels apply to structures which are correctly planned and built, i.e. without systematic errors.

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 mooring line.

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

2.2.3 Robustness

2.2.3.1 Robustness against possible systematic errors is desirable and structures should be designed with this in mind.

2.2.3.2 The less mature the technology, the larger is the need for robustness, e.g. in terms of redundancy. It is recommended to consider introducing such robustness in the case of floater concepts and technologies which are not proven or are otherwise immature.

2.3 Design principles and design conditions

2.3.1 Methods for structural design

2.3.1.1 In this standard, the following design principles and design methods for limit state design of floating wind turbine structures are considered applicable:

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

Application of other methods such as WSD is in general not foreseen. However, for limit state design of cables, WSD is used as per custom.

2.3.1.2 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.

2.3.1.3 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.

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

2.3.2 Aim of the design

2.3.2.1 Structures and structural elements shall be designed to:

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

2.3.3 Design conditions

2.3.3.1 The design of a structural system, its components and details shall satisfy the following requirements:

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

Except for shell structures, which are not regarded as ductile, structures and structural components shall possess ductile behaviour unless the specified purpose requires otherwise.

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

2.4 Limit states

2.4.1 General

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

2.4.1.2 The following limit states are considered in this standard:

— Ultimate limit states (ULS) corresponding to the maximum load-carrying resistance.
— Fatigue limit states (FLS) corresponding to failure due to the effect of cyclic loading.
— Accidental limit states (ALS) corresponding to survival conditions in a damaged condition or in the presence of abnormal environmental conditions.
— Serviceability limit states (SLS) corresponding to project-defined criteria applicable to intended use.

2.4.1.3 Examples of some, but not all, limit states within each category:

ULS:

— loss of structural resistance (excessive yielding and buckling)
— failure of components due to brittle fracture
— loss of static equilibrium of the structure, or of a part of the structure, considered as a rigid body, e.g. overturning or capsizing
— failure of critical components of the structure caused by exceeding the ultimate resistance (which in some cases is reduced because of repetitive loading) or the ultimate deformation of the components
— excessive deformations caused by ultimate loads

— transformation of the structure into a mechanism (collapse or excessive deformation)

FLS:

— cumulative damage due to repeated loads.

ALS:

— structural damage or failure caused by accidental loads
— exceedance of ultimate resistance of damaged structure
— maintain global structural integrity after local damage or flooding.

SLS:

— displacements or rotations that may alter the effect of the acting forces
— displacements or rotations that may change the distribution of loads between supported rigid objects and the supporting structure
— excessive vibrations and accelerations producing discomfort or affecting non-structural components
— motions that exceed the limitation of equipment
— temperature-induced deformations
— deformations or movements that effect the efficient use of structural or non-structural components or the operation of the turbine.
— deformations or crack widths which effect watertightness
— corrosion that reduces the durability of the structure and affects the properties and geometrical parameters of structural and non-structural components.

2.4.1.4 Accidental limit states with a probability of occurrence of less than 10^{-3} per year and involving only a single floating wind turbine unit should be considered as an SLS, i.e. it is up to the owner of the unit to improve the structural integrity if relevant from an economical or reputational viewpoint. Accidental limit states involving progressive failure or failure with high economical or societal impact shall always be considered.

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 for the load and resistance factors are set such that possible unfavourable realizations 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, and 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.

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

2.5.2.2 There are two approaches to establish the design load effect S_{di} associated with a particular load F_i:

(1) The design load effect S_{di} is obtained by multiplication of the characteristic load effect S_{ki} by a specified load factor γ_{fi}:

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

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

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

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

Approach (1) shall be used to determine the design load effect when a proper representation of the dynamic response is the prime concern, whereas approach (2) shall be used if a proper representation of nonlinear material behaviour or geometrical nonlinearities or both are the prime concern.

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:

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

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 among the load combinations that are specified to be investigated in order to obtain a characteristic combined load effect S_k equal to a particular quantile in the distribution of the true combined load effect S.

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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 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 in accordance with [2.5.2.3]. For use in design, the design combined load effect S_d is selected as the most unfavourable value among the design combined load effects that result for these specified sets of characteristic load effects.

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

Guidance note:

The aerodynamic 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 aerodynamic damping and a correct separate wave load effect in a structural analysis for the wave load, then the structure may need to be analysed for the sought-after combined load effect for a simultaneous application of the wind load process and the wave load process.

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2.5.2.7 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 γ_m:

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

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

$$R_d = R(\sigma_d)$$

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

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

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

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

2.5.2.10 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 ALS, the characteristic load effect is a specified value, dependent on operational requirements.
— 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

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

2.5.5 Load and resistance factors

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

2.6 Design assisted by testing

2.6.1 General

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

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

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

2.6.2 Model tests

2.6.2.1 For novel designs, model tests shall be carried out. Model tests can be used to validate software, to check effects which are known not to be adequately covered by the software, and to check the structure if unforeseen phenomena could occur. The tests shall be as realistic as possible with respect to scaling of wind, wave and current loading, considering issues such as scaling laws and inadequate model test basins. Also, to make the tests as realistic as possible and obtain correct wind forces, it may be necessary to properly represent the effect that the wind turbine control system has on the wind forces. Also, a correct representation of the turbulence spectrum and spatial coherence of the wind will in most cases be important.

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

2.6.3.1 Full-scale tests or monitoring of existing structures may be used to give information on response and load effects to be utilized in updating and refinement of structural design procedures. The experience from a full-scale test of an existing similar structure may provide an updated and more accurate prediction of the response and thereby imply an improved control of the achieved safety level. The possibility of obtaining simultaneous measurements on several floating units in a wind farm should be explored with a view to improving the data to be used for design.

2.6.3.2 Full-scale tests are important and necessary in order to achieve an optimal design and it is therefore recommended to carry out such tests.

2.7 Probability-based design

2.7.1 General

2.7.1.1 Probability-based design, based on a full probabilistic representation of governing load and resistance variables in terms of their respective probability distributions, is an option which forms an alternative to design by the partial safety factor method. When such probability-based design is opted for, the design shall be carried out to meet a safety level which is expressed in terms of a failure probability and shall be set equal to the nominal annual target failure probability as specified in [2.2.2.1] for the required consequence class.

2.7.1.2 The target safety level applies to design of an entire structure and will in practice also be the safety level for individual failure modes, since one failure mode is usually dominating. In the calculation of the failure probability which is to be checked against the target failure probability, it is important to consider possible system effects, for example failure of the weakest link of a long mooring line.

2.7.1.3 For probability-based design, see to specifications and requirements given in DNVGL-ST-0126.

SECTION 3 ENVIRONMENTAL CONDITIONS

3.1 Introduction

3.1.1 Definition

3.1.1.1
Environmental conditions consist of all natural phenomena which may influence the design of a floating wind turbine structure by governing its loading, its capacity or both.

3.1.1.2
Environmental conditions cover virtually all natural phenomena on a particular site, including but not limited to meteorological conditions, oceanographic conditions, water depth, ground 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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3.1.1.3
The requirements for environmental conditions given in DNVGL-ST-0437 apply to floating wind turbine structures with the exceptions, deviations and additions in this section. Useful guidance regarding environmental conditions is given in DNVGL-RP-C205.

3.1.2 Design to environmental classes

3.1.2.1 Environmental conditions are in principle site-specific, i.e. they refer to a specific location; a particular point in space. However, for floating wind turbine structures which are expected to be mass produced units, design to some specified environmental class rather than to a particular location can be foreseen. This is based on the idea that many locations with fairly similar environmental conditions can be grouped within some defined environmental class, and a structure designed to this class can then subsequently, on a case-by-case basis, be qualified for application on a particular location. It should be noted that while a floating substructure may be designed for an environmental class, the station keeping system is expected to be designed for a specific site.

3.1.2.2 This standard does not specify any predefined environmental classes. The standard provides guidance in terms of regional environmental data, which can be used as a basis for defining environmental classes, see [3.6].

3.1.2.3 An environmental class can be formed by the environmental conditions which prevail within a particular region and which can be considered the same throughout the entire region. An environmental class can be considered representative for more than one particular region when the regions in question share the same or similar environmental conditions.

3.1.2.4 Since the relative proportion between magnitudes of wind and waves will in general not be the same in two different regions, an environmental class to be specified so as to be applicable for design of floating wind turbine structures for use in two or more different regions will in practice have to be defined based on an envelope of all wind and wave climates encountered in those regions. For some or all regions intended to be covered by a specified environmental class, some of the specified environmental properties of the environmental class may therefore appear stricter than the actual values of these properties within these regions.

3.1.2.5 For fatigue analysis and fatigue design, the concurrent combination of wind and waves is essential and for certain systems and/or certain responses current is also important. The specification of an environmental class shall therefore include specification of the possible dependencies between the following quantities:

— significant wave height
— peak wave period (or zero-upcrossing period)
— mean wave direction
— directional spreading of waves
— 10-minute mean wind speed (or mean wind speed with other averaging period than 10 minutes) in specified reference height
— standard deviation of wind speed
— mean wind direction
— wind shear
— misalignment between wind and wave directions
— current speed and direction (if relevant).

e.g. in terms of a scatter diagram for these quantities. Vortex induced motions (VIM) and vortex induced vibrations (VIV) may increase the wind and wave loads and should therefore be considered in this context. The effect of turbine wakes and in particular low-frequency meandering wakes should also be considered, see IEC 61400-1.

3.2 Wind, waves and current

3.2.1 General

3.2.1.1 This subsection gives requirements which come in addition to those given in DNVGL-ST-0437 regarding environmental conditions and which shall be fulfilled when floating structures are to be used to support offshore wind turbines.

3.2.1.2 Simultaneous wind, wave and current data are important to allow for time domain analyses that may be necessary in order to carry out fatigue analyses.

3.2.1.3 As part of the specification of an environmental class or of site-specific environmental data, the correlation between wave data and wind data for use in fatigue design shall be established. This correlation can be expressed in terms of the joint long-term probability distribution for the significant wave height H_S, the peak period T_P (or alternatively the zero-upcrossing period T_Z), the 10-minute mean wind speed U_{10}, the standard deviation of the wind speed σ_U, the mean wave direction and the mean wind direction and can for practical purposes be represented in the form of a scatter diagram.

Guidance note:

The scatter diagram is used in fatigue design to establish a fatigue load history and in particular fatigue stress histories in critical components and cross sections. It is important to include mean wave direction and mean wind direction in the scatter diagram in addition to the significant wave height and the mean wind speed. This is so, because these directions are of great importance for the stress response as a result of the sensitivity in the aerodynamic damping to misalignment between wind and waves. For most modes of motion, aerodynamic damping is known only to be significant when the wave direction is in line with the rotor axis, which in turn is usually aligned with the mean wind direction. For yaw motions, aerodynamic damping remains high regardless of the direction of wind and waves.

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3.2.1.4 As part of the specification of an environmental class or of site-specific environmental data, the distributions of the wave energy content and the wind energy content over frequencies shall be established. In the short term, these distributions can be represented by the power spectral densities for waves and wind, respectively, conditioned on the relevant environmental parameters referenced in [3.2.1.3]. Adequate models for power spectral densities for waves and for wind are given in DNVGL-RP-C205.

3.2.1.5 A floating support structure is a more compliant system than a fixed support structure. An adequate representation of dynamics may require a more thorough and improved representation of simultaneous wind, waves and current than the one which is currently given in DNVGL-ST-0437.

3.2.2 Wind

3.2.2.1 The wind climate is represented by the 10-minute mean wind speed U_{10} and the standard deviation σ_U of the wind speed, both referring to a specified reference height. In the short term, i.e. over a 10 minute period, stationary wind conditions with constant U_{10} and constant σ_U are assumed to prevail.

Guidance note:

The specified reference height for the wind speed can be the hub height or any other height in which wind data happen to be recorded. A reference height of 10 m is commonly used.

The 10-minute mean wind speed U_{10} is a measure of the intensity of the wind. The standard deviation σ_U is a measure of the variability of the wind speed about the mean. When special conditions are present, such as when tornados, cyclones and typhoons occur, a representation of the wind climate in terms of U_{10} and σ_U may be insufficient.

Mean wind speeds based on other averaging periods than the 10 minutes referenced here can in principle be used for representation of the wind climate instead of the 10-minute mean wind speed U_{10}, for example the 1-hour mean wind speed. The 10 minute mean wind speed U_{10} is quoted because wind data are usually available in terms of this particular mean wind speed.

Appropriate conversions of the 10 minute mean wind speed to mean wind speeds with other averaging periods need to be considered and implemented when combining wind data with wave data with other reference durations, such as 3 or 6 hours, and when performing simulations with other simulation lengths than 10 minutes.

Compliant moorings feature natural periods which can be in excess of 200 seconds, especially in deep waters. In such cases, representing the wind speed in terms of the 10 minute mean wind speed will be insufficient and conversion to mean wind speeds with longer averaging times will be necessary.

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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 σ_U.

3.2.2.3 The turbulence intensity is defined as the ratio σ_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 σ_U and expresses how the energy of the wind speed is distributed between various frequencies.

3.2.2.5 The wind energy content in relevant sea states and its distribution over frequencies shall be considered. The power spectral density is useful for this purpose. Various power spectral density models exist, usually expressing the power spectral density in terms of parameters such as the mean wind speed with some averaging period, for example the 10-minute mean wind speed U_{10}, and the standard deviation σ_U of the wind speed. Because U_{10} and σ_U vary with height above sea level, the power spectral density will also be a function of this height. For useful power spectral density models, see DNVGL-RP-C205, which also provides recommendations for the integral length scale that constitutes an important property of any power spectral density model.

3.2.2.6 For selection of appropriate power spectral density models for design of floaters it is of utmost importance to make sure that the models are valid for wind over water and that they provide a good representation not only in the high frequency range but also in the low frequency range.

3.2.2.7 The correlation between wind speeds at separated points in space is of importance and needs to be modelled. Coherence spectra can be used for this purpose. Different coherence spectra may apply for downwind and for crosswind. Different integral length scales may apply for longitudinal, lateral and vertical separations. Models for coherence spectra are provided in IEC 61400-1.

Guidance note:

Also DNVGL-RP-C205 may be considered, but it should be noted that some of the turbulence models referred to therein are not applicable for design of wind turbines.

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3.2.2.8 It shall be ensured that the representation of the wind in the low frequency range is adequate. This includes, but is not limited to, an adequate representation of power spectral density in the low frequency range as well as adequate models for representation of gust events.

Guidance note:

The requirement reflects that for floating support structures, low frequency motion components and response components are expected.

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3.2.2.9 A number of design load cases proposed in DNVGL-ST-0437 involve an extreme operating gust (EOG) which is one of a number of defined reference wind conditions used in DNVGL-ST-0437 for definition of load cases. The EOG event presently specified in DNVGL-ST-0437 is based on a duration of 10.5 sec and is developed for design of bottom-fixed wind turbines and their support structures. With its duration of 10.5 sec, this EOG is inadequate for design of most floating support structures. However, for design of the tower, the EOG with 10.5 sec duration may still be relevant and should always be considered, even when the floating wind turbine unit have higher natural periods.

3.2.2.10 One or more gust events with longer durations than 10.5 sec shall be defined and shall be used in design. The gust events shall cover events which can be expected and which shall be about equally likely to occur as the EOG. The gust events shall reflect the needs in design under due consideration of the frequencies encountered for the dynamics of the floating unit. Gust characteristics to consider in this context include, but are not limited to, duration of gust event, maximum wind speed, and rise time of wind speed to maximum. In particular for large rotors, the spatial correlation of the wind field is an important issue to consider when appropriate gust events for use in design are to be defined. The gust events shall be used instead of the EOG in those proposed design load cases that are defined in terms of the EOG in DNVGL-ST-0437. The natural periods of the floating wind turbine unit (typically in the range from 10 to 100 sec) shall be considered when selecting the durations of the gusts, i.e. the durations of the gusts shall be selected where the largest dynamic responses are expected.

Guidance note:

Unless data indicate otherwise, the following gust model may be used as a basis for defining gust events with longer durations than 10.5 seconds:

$$V(z,t) = \begin{cases} u(z) - 0.37 \cdot V_{gust} \cdot \sin(3\pi \cdot t / T) \cdot (1 - \cos(2\pi \cdot t / T)) & for\ 0 \le t \le T \\ u(z) & otherwise \end{cases}$$

where:

t = time,

T = duration,

z = height,

$u(z)$ = wind profile,

V_{gust} =reference wind speed = difference between maximum and minimum wind speed during gust.

The duration and reference wind speed of the gust event should be carefully chosen with a view to the expected dynamics of the floating unit and such that the resulting load cases from combining the gust events with specified fault situations come out with return periods of approximately 50 years. It is noted in this context that gust models as the present are always artificial models which may favour or disfavour certain floater concepts.

The reference wind speed V_{gust} may be taken as:

$$V_{gust} = \min \left\{ 1.35 \cdot (V_{e1} - V_{hub}); (0.9 \cdot \ln(T) + 1.18 \cdot (\frac{\sigma_1}{1 + 0.1 \cdot (\frac{D}{\Lambda_1})}) \right\}$$

where:

V_{e1} = extreme 3 second wind speed at hub height with a return period of one year

V_{hub} = 10 minute mean wind speed at hub height

D = rotor diameter

σ_1 = turbulence standard deviation for normal turbulence model (NTM), see DNVGL-ST-0437

Λ_1 = turbulence scale parameter = 0.7z if z < 60 m, otherwise equal to 42 m.

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3.2.2.11 One or more gust events defined in terms of sudden directional changes of the wind shall be considered in design.

Guidance note:

The duration of the directional change should be selected in similar way as described in [3.2.2.9] and [3.2.2.10]

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3.2.3 Waves

3.2.3.1 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.

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3.2.3.2 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.

3.2.3.3 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.

3.2.3.4 The wave crest height H_C is the height of the highest crest between two successive zero-upcrossings of the sea elevation process.

3.2.3.5 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 process is distributed between various frequencies.

3.2.3.6 The wave energy content in the relevant sea states and its distribution over frequencies shall be considered. The power spectral density is useful for this purpose. Various power spectral density models exist, usually expressing the power spectral density in terms of sea state parameters such as the significant wave height H_S and the peak period T_P, where the peak period T_P is somehow related to the zero-upcrossing period T_Z. For useful power spectral density models, reference is made to DNVGL-ST-0437 and DNVGL-RP-C205.

Sea states dominated by wind-generated waves can usually be represented by one-peaked power spectral density models. When swell components can be expected in addition to wind-generated waves, these have to be properly represented in the power spectral density model, where they may show up as a second peak. Caution must be exercised when modelling the power spectral density, because power spectral density models are often site-specific in that they are developed based on site-specific data.

3.2.3.7 The JONSWAP wave spectrum recommended in DNVGL-ST-0437 for representation of the power spectral density of wind-generated waves may be insufficient for floating wind turbine structures, because floating wind turbine structures can be excited in heave, roll and pitch by swells of 20 to 25 seconds period. For floating wind turbine structures which can be excited by swells and which are to be designed to an environmental class which includes swells, a two-peaked power spectrum model shall be used for representation of the power spectral density. The Torsethaugen spectrum is one such two-peaked power spectrum model, developed for North Sea conditions; see DNVGL-RP-C205. Alternatively two JONSWAP spectrums can be combined to represent wind-generated waves and swell, possibly also with different directions.

3.2.3.8 At locations where wind-generated waves and swells occur concurrently, the swell components can have a very different direction than the direction of the wind-generated waves.

3.2.3.9 The 50 year wave height is often estimated as a factor times the 50 year significant wave height. A realistic value for this factor shall be assumed in design. In the deep waters that floating wind turbine structures usually are to be designed for, this factor, the ratio between the 50-year wave height and the 50-year significant wave height, may reach a value of about 2.0.

Guidance note:

The ratio between the mode of the largest of 1000 Rayleigh-distributed wave heights and the significant wave height in a 3 hour stationary sea state is 1.86 and is often used as a rough estimate of the ratio between the 50 year wave height and the 50 year significant wave height. IEC 61400-3 recommends use of 1.86 as an estimate for the ratio between the 50 year wave height and the 50 year significant wave height. It is noted that this recommendation may be non-conservative in deep waters.

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3.2.3.10 The potential for earthquake-induced sea waves, also known as tsunamis, shall be assessed. Likewise, the potential for tsunami-like waves caused by underwater landslides, which are not necessarily initiated by earthquakes, shall also be assessed.

Guidance note:

Tsunamis are seismic sea waves. Tsunamis have very long periods and behave like shallow water waves even when passing through deep parts of the ocean.

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3.2.4 Current

For modelling of current, see DNVGL-RP-C205. For vortex-induced vibrations and vortex-induced motions see DNVGL-RP-C205.

3.3 Water depth and water level

3.3.1 Water depth

3.3.1.1 A range of water depths shall be defined as part of the definition of an environmental class.

3.3.2 Water level

3.3.2.1 The water level consists of a mean water level in conjunction with tidal water and a wind- and pressure-induced storm surge. The mean water level (MWL) is the long-term average water level, i.e. the arithmetic mean of all sea levels measured over a long period. The still water level (SWL) is the average water surface elevation at any instant, excluding local variations due to waves, but including the effects of tides and storm surges. The tidal range is defined as the range between the highest astronomical tide (HAT) and the lowest astronomical tide (LAT), see Figure 3-1. The mean water level (MWL) is approximately 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.

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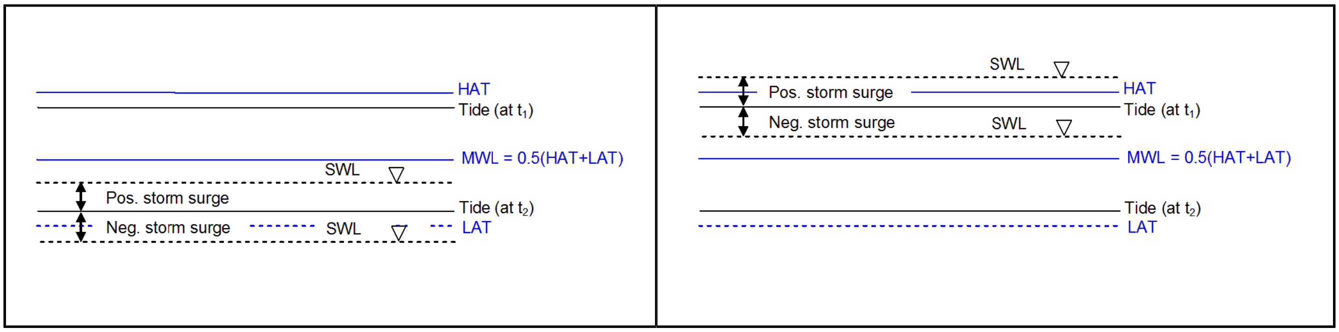

Figure 3-1 Definition of water levels when low still water level is governing (left) and when high still water level is governing (right)

3.3.2.2 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.

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3.3.2.3 Set-down effects associated with tendons may require special focus on water level with consideration of effects of tides, storm surges and tsunamis.

3.4 Seismicity

3.4.1 Assessment

3.4.1.1 The level of seismic activity of the area where the floating wind turbine structure is to be installed shall be assessed on the basis of previous records of earthquake activity as expressed in terms of frequency of occurrence and magnitude. For areas where detailed information on seismic activity is available, the seismicity of the area may be determined from such information. For areas where detailed information on seismic activity is not available, the seismicity shall be determined on the basis of detailed investigations, including a study of the geological history and the seismic events of the region.

3.4.1.2 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 recognized industry practice.

3.4.2 Design aspects

3.4.2.1 For details of seismic design criteria, see DNVGL-ST-0437 and ISO 19901-2.

3.4.2.2 Seismicity shall be considered in the design, if relevant. Assessment of seismicity can be of significant importance for design of tension leg platforms. The sensitivity to earthquake is related to which modes of motion are restrained. All types of floaters may be impacted either as a function of ground motions on the anchors or by associated sudden wave loading.

3.4.2.3 Assessment of effects of tsunamis caused by earthquakes can be critical for the design of station keeping systems.

Guidance note:

In deep waters, tsunami wave crests will usually be so small that they will be virtually undetectable to ships. As the tsunami travels from deep waters to shallower waters, the wave crest becomes amplified. The effect of tsunamis is therefore related to water depth. The effect of tsunamis is also related to which modes of motions are restrained.

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3.5 Ground conditions

3.5.1 General

3.5.1.1 Ground conditions consist of soil conditions or rock conditions or both. For design of floaters whose structural design depends on the ground conditions, a range of ground conditions shall be defined as part of the definition of an environmental class for the design of the floater.

3.5.1.2 For design of station keeping systems and their components, such as anchors and mooring lines, a range of ground conditions shall be defined as part of the definition of an environmental class for the design. For each particular site-specific wind farm project, the design of these station keeping systems and their components shall be qualified for application on the actual site.

3.5.1.3 For qualification of the designs of floating wind turbine structures and their station keeping systems for use in a wind farm on a particular site, ground conditions shall be established in those positions where the station keeping systems of the floating structures are anchored or otherwise transfer the floater loads to the seabed soils. These positions are referred to as foundation positions and they are not necessarily the positions of the floating structures themselves as there may be anchor points that are located some distance away from the floater.

3.5.1.4 Typical ranges of soil parameters for soils classified as cohesionless, i.e. sand, are given in Table 3-1.

Table 3-1 Typical geotechnical parameters for sand

Soil type	*Friction angle φ*	*Submerged unit weight γ' (kN/m^3)*	*Poisson's ratio ν*	*Void ratio e*
Loose	28 – 30º	8.5 – 11.0	0.35	0.7 – 0.9
Medium	30 – 36º	9.0 – 12.5	0.35	0.5 – 0.8
Dense	36 – 41º	10.0 – 13.5	0.35	0.4 – 0.6

3.5.1.5 Typical ranges of soil parameters for soils classified as cohesive, i.e. clay, are given in Table 3-2.

Table 3-2 Typical geotechnical parameters for clay

Soil type	*Undrained shear strength su (kN/m^2)*	*Submerged unit weight γ' (kN/m^3)*	*Poisson's ratio ν*	*Void ratio e*
Very soft	< 12.5	4 – 7	0.45	1.0 – 3.0
Soft	12.5 – 25	5 – 8	0.45	0.8 – 2.5
Firm	25 – 50	6 – 11	0.45	0.5 – 2.0
Stiff	50 – 100	7 – 12	0.45	0.4 – 1.7
Very stiff	100 – 200	10 – 13	0.45	0.3 – 0.9
Hard	> 200	10 – 13	0.45	0.3 – 0.9

3.5.1.6 When developing a site-specific floater design, site-specific ground investigations shall form the basis for establishing ground conditions for a particular wind farm project. For 'type' design floaters where a particular site is not specified, but the anchoring solution is included, the assumed ground conditions shall be specified.

For soil investigations, the requirements and recommendations given in DNVGL-ST-0126 and DNVGL-RP-C212 apply. For definition and estimation of characteristic soil properties, see DNVGL-RP-C207 and DNVGL-RP-C212.

3.6 Regional data for definition of environmental classes

3.6.1 General

3.6.1.1 Mass production of floating wind turbine structures is foreseen with design to some environmental class rather than to site-specific environmental conditions. The idea is that many locations with approximately the same metocean conditions can be grouped together within a defined environmental class which is then used as basis for design of a unit for mass production. Subsequently, qualification of such a mass produced unit for application on a particular location can then be carried out on a case-by-case basis.

3.6.1.2 For definition of environmental classes, key environmental wave parameters, based on wind-wave correlation, may be checked against the data given for various regions in the world in DNVGL-RP-C205 App.C. The significant wave heights tabulated in DNVGL-RP-C205 refer to 3-hour stationary sea states. For stationary sea states with duration other than three hours, the significant wave height data in DNVGL-RP-C205 must be suitably converted to properly refer to the actual sea state duration, see [3.6.1.3]. The data in DNVGL-RP-C205 App.C, include Weibull scale and shape parameters for regional long-term distributions of significant wave height.

3.6.1.3 The significant wave heights with specified return periods given in DNVGL-RP-C205 App.C refer to significant wave heights in stationary sea states of duration 3 hours. The shorter the duration of a stationary sea state, the higher is the significant wave height with a specific return period. It may be of interest to convert the significant wave height associated with a 3-hour stationary sea state duration to a significant wave height associated with a stationary sea state duration T_S and at the same time retain the associated return period. This can be done by application of an appropriate conversion factor, which depends on the desired stationary sea state duration T_S as well as on the return period to be retained.

Guidance note:

When the upper tail of the long-term probability distribution of the significant wave height can be represented by a Weibull distribution and when the duration of a stationary sea state is T_S, then the following relationship can be assumed between the significant wave height H_S of the sea state and its associated return period T_R

$$\exp\left(-\left(\frac{H_S}{\alpha}\right)^{\beta}\right) = \frac{1}{N \cdot T_R}$$

in which N is the number of stationary sea states of duration T_S in one year. For T_S=3 hours, the number of stationary sea states in one year is N=2922.

The coefficients α and β can be solved from the two equations that can be established from this relationship based on two sets (T_R, $H_S(T_R)$), for example the values of H_S for the two return periods 50 and 500 years.

Based on the value of β, the significant wave height H_{Snew} in a stationary sea state of duration T_{Snew} can be calculated from the significant wave height H_S in a stationary sea state of duration T_S in accordance with the following expression:

$$H_{Snew} = H_S \cdot \left(1 + \frac{\ln\left(\frac{T_S}{T_{Snew}}\right)}{\ln(N \cdot T_R)}\right)^{1/\beta}$$

in which N is the number of stationary sea states of duration T_S in one year and T_R is the specified return period which is retained by the conversion. T_R must be given in units of years.

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3.6.1.4 The distribution of the zero-upcrossing period T_Z, conditioned on the significant wave height H_S, can be represented by a lognormal distribution. The mean value of T_Z conditioned on H_S is expressed as:

$$E[T_Z \mid H_S] = \exp(\mu + \frac{1}{2} \cdot \sigma^2)$$

and the standard deviation of T_Z conditioned on H_S is expressed as:

$$D[T_Z \mid H_S] = E[T_Z \mid H_S] \cdot \sqrt{\exp(\sigma^2) - 1}$$

in which

$$\mu = E[\ln T_Z] = 0.70 + a_1 \cdot H_S^{a_2}$$

$$\sigma = D[\ln T_Z] = 0.07 + b_1 \cdot \exp(b_2 H_S)$$

These expressions require H_S to be given in units of metres and T_Z to be given in units of seconds. Values for the coefficients a_1, a_2, b_1 and b_2 are given for various regions in DNVGL-RP-C205.

3.6.1.5 For definition of environmental classes, it is important to address the wave energy content in the relevant sea states and its distribution over frequencies. The power spectral density is useful for this purpose. See [3.2.3.6].

3.6.1.6 For definition of environmental classes, key environmental wind parameters for various regions in the world are given in DNVGL-RP-C205.

3.6.1.7 For definition of environmental classes, it is important to address the wind energy content in the relevant sea states and its distribution over frequencies. The power spectral density is useful for this purpose. See [3.2.2.5].

3.6.1.8 For definition of environmental classes it is important to consider the correlation between wind and waves. This can be expressed in terms of a correlation between the 10-minute mean wind speed U_{10} and the significant wave height H_S. In the case of wind driven waves a positive correlation can be expected and should be considered, however, a phase difference between U_{10} and H_S can be expected. For definition of environmental classes it is also important to consider the wind direction and the wave direction as well as the directional scatter and the possible misalignment between wind and waves. These matters are of particular importance for fatigue design. It may be important to consider collinear wind and waves part of the time and misaligned wind and waves the rest of the time. In case assumptions in this respect have to be made owing to lack of data, it may vary between different types of structures which assumptions will be conservative and which will be nonconservative.

3.6.1.9 The regional environmental data given in the tables in DNVGL-RP-C205 and referenced in [3.6.1.2] through [3.6.1.6] are not site-specific data and cannot be used for detailed site-specific design. There may be large variations from site to site within a region. The data, being indicative only, are to be used only for definition of environmental classes. All other use is limited to concept studies in an early phase.

3.6.1.10
Information on statistical distribution of currents and their velocity profile is generally scarce for most areas of the world. For definition of environmental classes, it is recommended to make conservative definitions of current velocities and profiles. Current in both alignment and misalignment with wind and waves should be included in the definition of the environmental class.

3.7 Other site conditions

3.7.1 Salinity

The salinity of the seawater shall be addressed as a parameter of importance for the design of cathodic protection systems. The salinity also has an impact on the buoyancy and consequently on the stability of the floating structure.

3.7.2 Temperature

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

3.7.2.2 Both air and seawater temperatures shall be considered when describing the temperature environment.

3.7.3 Marine growth

Marine growth on structural components in water and in the splash zone, caused by plant, animal and bacteria life, shall be considered and accounted for in design as specified in DNVGL-ST-0437. In accounting for marine growth it is important to consider its effect in terms of its weight in water as well as its effect in terms of increased dimensions of the affected structural members.

Marine growth shall be based on an assessment of site specific conditions. Marine growth shall be considered for structures and power cables.

SECTION 4 LOADS AND LOAD EFFECTS

4.1 Introduction

4.1.1 General

4.1.1.1 In this section, loads, load components and load combinations to be considered in the overall strength analysis for design of floating support structures for wind turbines are specified. Requirements for the representation of these loads and their combinations as well as their combined load effects are given.

4.1.1.2 Determination of aero- and hydrodynamic loading and response is vital, since this loading and response involve wind excitation, wave excitation, added mass, aerodynamic damping, wave damping and structural damping, and the stiffness as well as the geometry of the floater in question. All these parameters are decisive for determination of wave frequency (WF), low frequency (LF) and high frequency (HF) floater motions. All these motion components are of importance and must be determined carefully. In this context, the influence of the turbine controller on the aerodynamic loads and in particular on the aerodynamic damping needs to be considered. For further details, reference is made to DNVGL-RP-C205 and DNVGL-RP-F205.

Guidance note:

Careful assessment of the combination of aerodynamic damping, wave damping and structural damping under different wind and wave directions and in particular under wind and wave misalignment is important for determination of the floater motions, since aerodynamic damping is known only to be significant when the wave direction is in line with the rotor axis, which in turn is usually aligned with the mean wind direction.

The mean wave direction can deviate significantly from the mean wind direction. In the case of swells, originating in different areas and being uncorrelated with the local winds, this is obvious. In the case of wind-generated waves, this may for example take place immediately after a change in the wind direction.

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4.1.2 Extreme loads

4.1.2.1 Extreme loading of floating support structures will be a larger challenge than extreme loading of bottom-fixed structures, because the wind will imply global motions that in turn will cause forces and stresses in the structure. Operational conditions during power production will often produce the governing extreme loads which will then be dominated by the thrust force formed by the wind loads. The largest thrust force usually occurs at the rated wind speed, but will sometimes occur at other wind speeds. Dynamic loads at wind speeds above the rated wind speed are controller dependent. The largest extreme loads may not always be from normal power production. Transients due to emergency stops may for example produce the highest tower base bending moments. Storm conditions with high waves, during which the turbine typically is parked, may also cause governing extreme loads due to wave motion in combination with high wind loads on the tower and RNA.

4.1.2.2 All load conditions, both during power production and in the parked condition for the turbine, need to be included in a full long-term analysis for the extreme loads.

4.1.3 Fatigue loads

Fatigue damage of floating support structures will be a larger challenge than fatigue damage of bottom-fixed structures, because the wind will imply global motions that in turn will cause forces and stresses in the structure. The wind force is essential to the fatigue behaviour of floating support structures owing to its significant contribution to the bending stresses in the various structural members.

4.1.4 Transportation loads

Transportation of floating wind turbine structures may give rise to load cases, which shall be considered in design.

4.1.5 Wind turbine loads

4.1.5.1 Rotational sampling, consisting of the wind experienced by a rotating point on the rotor and resulting from spatial variation of the turbulence over the swept area of the rotating turbine, will lead to transfer of the low frequency part of the turbulence to loads at frequencies equal to multiples of the rotational frequency of the rotor. This is of importance for the wind turbine cyclic loading and shall be considered in design.

4.1.5.2 Yaw loads resulting from a tilted rotor during pitch of the floater shall be considered in design, as they can be important for a floater with low yaw resistance such as a spar. Yaw loads caused by turbulent crosswind shall also be considered.

4.1.6 Wake-induced loads

4.1.6.1 Loads induced by wakes behind upstream wind turbines in wind farms shall be considered in design.

4.1.6.2 Within wind farms low-frequency turbulence may occur and cause sideways oscillations of wakes which may or may not hit downstream wind turbines and their towers. The possible consequences of such “meandering” wakes for floating units in terms of associated load cases should be considered in design.

4.2 Response characteristics

4.2.1 General

4.2.1.1 The response characteristics of a floating support structure are of interest in the design of the structure. As guidance in design, this subsection provides an overview of response characteristics for various floater concepts of relevance for support of floating wind turbines.

4.2.1.2 With reference to their compliant and “soft” behaviour in the horizontal plane, most floater types typically have natural periods in surge, sway and yaw which are longer than 100 sec and which are governed by the station keeping system.

4.2.1.3 Depending on the location and the sea state, ocean waves contain substantial energy in the spectral period range 5 to 25 sec. For a floater, the natural periods of motion are key features, which reflect the design philosophy applied for the floater. The natural period in heave is normally above 25 sec for a spar and below 5 sec for a TLP.

4.2.1.4 Roll and pitch periods may be more important than heave periods; however, what is most important is to avoid coupled vibration forms which may occur if heave periods are too close to roll and pitch periods. Coupling effects between all six DOFs may be important due the combined effect of rotor loads and the control algorithms. In particular, the coupling between the yaw and pitch modes of motion may be important for floaters with low yaw resistance, such as spars, since a tilted rotor will result in yaw loads.

4.2.2 Spars

Although spars are not restrained in heave, they are characterized by small heave motions owing to their large draughts. This is advantageous for power take-off cables, umbilicals and moorings. Solid and liquid ballast is often employed at the keel to control the floating performance. The dominant loads are wind and wave loads; however, DDFs do have a large area exposed to current forces. Strakes added to the hull of a spar may reduce possible vortex-induced cross-flow oscillations, but will at the same time increase the added mass and the drag forces on the spar. A check against VIM shall be carried out, see guidance in [7.1.3].

4.2.3 Semi-submersibles

Semi-submersibles are usually column-stabilized units consisting of support columns attached to submerged pontoons. The pontoons may be of different designs such as ring pontoons, twin pontoons and multi-footing pontoons. semi-submersibles have small water-plane areas which give rather high natural periods in vertical modes. The natural period in heave is usually outside the range of wave periods except in extreme sea states. This implies that a semi-submersible normally has relatively small vertical motions compared to a monohull floater such as a barge. However, its behaviour in extreme weather conditions requires flexible mooring and power take-off systems. A variety of mooring systems are available for the semi-submersible concept.

4.2.4 Tension-leg platforms

4.2.4.1 A tension-leg platform (TLP) differs fundamentally from other floater concepts in that it is the tendon stiffness rather than the water-plane stiffness that governs the vertical motions. The tension system is a soft spring in surge, sway and yaw, but a stiff spring in heave, roll and pitch.

4.2.4.2 A TLP generally experiences wave frequency (WF) motions in the horizontal plane that are of the same magnitude as those of a semi-submersible of comparable size. In the vertical plane, however, the TLP will behave more like a bottom-fixed structure with practically no WF motion response. WF forces are directly counteracted by the tendon stiffness forces. Tendon forces to keep the floater in position usually become large.

4.2.4.3 Higher order sum-frequency forces may introduce springing and/or ringing response in vertical modes, see DNVGL-RP-C205. These effects may give significant contributions to the fatigue responses of the tethers.

4.2.4.4 Set-down denotes the kinematic coupling between the horizontal surge/sway motions and the vertical heave motions. Set-down is important in the calculation of tether forces and in the calculation of responses in power take-off cables. Set-down can be of the same order of magnitude as wave-frequency surge and sway in shallow waters. For further details, reference is made to DNVGL-OS-C105.

4.2.5 Monohull structures

Because of large water plane areas and relatively small draughts, monohull structures are susceptible to large motions. A monohull structure can be shaped as a barge or as a ship. The combination of head sea and beam swell form a critical condition for a monohull structure. Significant accelerations in roll may occur for oblong structures such as ships and may have an impact on the turbine and on the design of cables and mooring system. Large bilge keels might be necessary in order to control the motions, regardless of the shape of the hull. Selection of a proper damping estimate is important for the prediction of responses.

4.3 Basis for definition of characteristic loads

4.3.1 Load categorization

4.3.1.1 Loads are categorized according to type. The load categorization is used as basis for definition of characteristic loads for use in design.

4.3.1.2 The following load categories are defined:

- permanent loads
- variable functional loads
- environmental loads
- accidental loads
- deformation loads
- pressure loads on hull
- abnormal wind turbine loads (loads associated with fault situations for the wind turbine).

Guidance note:

Abnormal loads are wind loads resulting from a number of severe fault situations for the wind turbine which result in activation of system protection functions. Abnormal wind loads due to fault conditions for the turbine have a higher probability of occurrence than accidental loads considered for the ALS. Depending on the type of fault, abnormal wind loads due to fault conditions for the turbine may even have a higher probability of occurrence than typical ULS loads.

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4.3.2 Definition of characteristic loads

4.3.2.1 Unless specific exceptions apply, as documented within this standard, the basis for selection of characteristic loads or characteristic load effects specified in [4.3.2.2] and [4.3.2.3] 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 steady conditions such as power production, idling and stand-still as well as transient conditions associated with start-up, shutdown, yawing and faults of the wind turbine.

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4.3.2.2 For the temporary design conditions, the characteristic loads and load effects in design checks shall be based on specified environmental design conditions as outlined in Table 4-1. The environmental design conditions shall be specified with due attention to the actual location, the season of the year and the consequences of failure. For design conditions during transport and installation, see DNVGL-ST-N001.

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.

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

	Limit states – temporary design conditions				
Load category	*ULS*	*FLS*	*ALS*		*SLS*
			Intact structure	*Damaged structure*	
Permanent (G)	Expected value				
Variable (Q)	Specified(1) value	Specified(1) load history	Specified(1) value		

	Limit states – temporary design conditions				
Load category	*ULS*	*FLS*	*ALS*		*SLS*
			Intact structure	*Damaged structure*	
Environmental (E); weather restricted	Specified value	Expected load history	Abnormal	Specified value	
Environmental (E); unrestricted operations[2]	Based on statistical data[3]	Expected load history		Based on statistical data[3,4]	
Accidental (A)			Specified value		
Deformation (D)	Expected extreme value	Expected load history	Specified value		
Prestressing (P)	Specified value				
(1) The specified value or the specified load history, as applicable, shall, if relevant, be justified by calculations. (2) See DNVGL-ST-N001. (3) See DNVGL-ST-N001, Sec. 3. (4) Joint probability of accident and environmental condition could be considered.					

4.3.2.3 For the operational design conditions, the basis for definition 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 definition of characteristic loads and load effects for operational design conditions

	Limit states – operational design conditions				
Load category	*ULS*	*FLS*	*ALS*		*SLS*
			Intact structure	*Damaged structure*	
Permanent (G)	Expected value				
Variable (Q)	Specified value				
Environmental (E)	98% quantile in distribution of annual maximum load or load effect (Load or load effect with return period 50 years)	Expected load history or expected load effect history	Abnormal	Load or load effect with return period not less than 1 year	Specified value
Accidental (A)			Specified value		
Abnormal wind turbine loads	Specified value	Expected load history			
Deformation (D)	Expected extreme value				
Prestressing (P)	Specified value				

Table 4-3 Statistical terms used for definition of characteristic loads and load effects

Term	*Return period (years)*	*Quantile in distribution of annual maximum*	*Probability of exceedance in distribution of annual maximum*
100-year value	100	99% quantile	0.01
50-year value	50	98% quantile	0.02
10-year value	10	90% quantile	0.10
5-year value	5	80% quantile	0.20
1-year value	–	Most probable highest value in one year, i.e. the mode in distribution of annual maximum	

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

4.4 Permanent loads (G)

4.4.1 Definition

4.4.1.1 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, including permanent pressure differences
— reactions to the above.

4.4.1.2 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.

4.4.1.3 Permanent pressures are dealt with together with other loads on the hull structure in [4.9].

4.4.2 Floater-specific issues

4.4.2.1 Pretension of tendons, used for station keeping of wind turbine floaters of the TLP type, shall be considered and treated as permanent loads. The pretension in tendons is influenced by tidal water. The most unfavourable water level shall be used in design.

4.4.2.2 Pretension of mooring lines constructed from steel wires and chains is usually a mean mooring line force dominated by the weight of the mooring line. Pretension of mooring lines shall therefore be categorized as a permanent load. Pretension of mooring lines constructed from fibre ropes is usually not dominated by weight, but is still reckoned as a permanent load.

Guidance note:

Fixed-length fibre ropes such as those used in taut mooring systems may experience relaxation over time, such that an initially high pretension in the beginning of a design window may become reduced as time elapses, resulting in a lower pretension at the end of this window. Fibre ropes used in tendons for TLPs will, over time, experience an analogous creep and corresponding elongation owing to pretension and the pretension may likewise become relaxed. Despite such time-dependent relaxation of pretension in fibre ropes, this pretension is still reckoned as a permanent load.

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4.4.2.3 The "as installed" mean tension in a mooring line should be obtained for verification of design assumptions.

Guidance note:

The mean tension is normally not measured directly, but can be obtained from measurements of line length and sag. The mean tension is very important to the fatigue loads in the mooring line as the stiffness may be very progressive with the tension level. The fatigue loads in the mooring line should be computed under due consideration of the mean load, considering the thrust for the turbine as well as any current loads.

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4.4.2.4 Permanent solid and liquid ballast used for floating stability purposes is categorized as a permanent load.

4.4.2.5 The influence of fabrication and installation tolerances shall be considered in the design of the floating structure. This is of particular relevance in taut mooring systems as the stretch of the mooring lines can be smaller than the installation tolerances. Also tendons may be sensitive to such tolerances.

4.5 Variable functional loads (Q)

4.5.1 Definition

4.5.1.1 Variable functional loads are loads which may vary in magnitude, position and direction during the period under consideration, and which are related to operation and normal use of the structure in question. Variable functional loads are often referred to as payload. For floating wind turbine structure, the variable functional loads mainly consist of:

— actuation loads
— loads on access platforms and internal structures such as ladders and platforms
— boat impacts from service vessels due to normal operation
— weight of variable ballast and pressures due to variable ballast
— crane operational loads.

4.5.1.2 The characteristic value of a variable functional load is the maximum or minimum specified value, whichever produces the most unfavourable load effects in the structure in question.

4.5.1.3 Boat impacts are dealt with in [4.5.2]. Pressures due to variable ballast loads are dealt with together with other loads on the hull structure in [4.9].

4.5.2 Boat impacts and collisions

4.5.2.1 Boat impact loads from approaching boats in normal operation are defined as variable functional loads. Boat impact loads from drifting boats are defined as accidental loads.

4.5.2.2 Boat landings, ladders and other secondary structures in and near the water line shall be designed against operational boat impacts in the ULS. The primary structure in and near the water line shall be designed against accidental boat impacts in the ALS. Furthermore, if an accidental boat 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 boat 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 the maximum authorized service vessel are not considered, see [4.5.2.6].

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

4.5.2.3 When primary structural parts such as the floater hull are exposed to boat impacts, these structural parts shall not suffer such damage that their capacities to withstand the loads they will 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 lose 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.

4.5.2.4 For design against operational boat impacts, the characteristic impact load shall be taken as the expected impact load caused by the maximum authorized 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 authorized 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 boat impact analysis. The expected impact load is part of the results from the risk analysis.

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

$$F = 2.5 \cdot \Delta$$

where F is the impact force in units of kN and Δ is the fully loaded displacement of the supply vessel in units of tons. This is based on an assumption of boat 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 DNVGL-RU-HSLC.

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.

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4.5.2.5 For design against accidental boat impacts, the characteristic impact load shall be taken as the impact load caused by unintended collision by the maximum authorized 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 authorized service vessel shall be considered.

Guidance note:

The maximum authorized service vessel is the largest expected vessel used in daily operation. Data for the maximum authorized 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.

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

4.5.2.6 Boat impact loads as described in [4.5.2.4] and [4.5.2.5] may need to be supplemented as necessary, for example if larger boats have to be considered in predicting loads from boat impacts due to normal operation. The design against boat impacts shall be based on the expected maximum authorized service vessel for the range of wind farms for which the floating support structure is meant to be used.

4.5.2.7 Boat impact loads on floating support structures need to be more thoroughly documented than for fixed structures. This is so because of the consequences of a boat collision, should the collision lead to penetration of a compartment wall and cause flooding, and because of the motions of two bodies with different motion characteristics. The consequences of a boat collision are governed by the ratio between the stiffness of the boat and the stiffness of the floater and by the ratio between the mass of the boat and the mass of the floater. The lateral compliance of the floating structure is no guarantee that the result of a boat collision will be less severe than for a bottom-fixed support structure, since the two objects can be in phase opposition which may render the result of the boat collision more severe than for a bottom-fixed structure. The wall thickness of floating structures may also be quite moderate as compared to that of a bottom-fixed structure.

4.5.2.8 Guidance for boat impact analysis is given in DNVGL-RP-C104 Sec.8. For how to perform advanced nonlinear analysis in design, see DNVGL-RP-C208 Sec.4.

4.6 Environmental loads (E)

4.6.1 Definition

4.6.1.1 Environmental loads are loads caused by environmental phenomena. These are loads whose magnitude and direction may vary with time, such as:

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

4.6.1.2 Environmental loads and load effects to be used for design shall be based on environmental data representative for the target region and relevant for the operation in question. Environmental loads and load effects to be used for design shall be determined by use of relevant methods applicable for the target region and for the operation of the structure, and taking into account the type, size and shape of the structure as well as its response characteristics.

4.6.1.3 Characteristic loads and load effects shall be determined as quantiles with specified probabilities of exceedance in the respective relevant probability distributions.

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

4.6.2 Floater-specific issues

4.6.2.1 In order to adequately capture effects associated with the natural frequencies of floating support structures when loads and responses are to be determined, a sufficient length of the involved simulations must be ensured. Simulation periods should therefore be increased from the typical duration of 10 minutes. A minimum of 3 hours is recommended to adequately capture effects such as nonlinearities, second order effects, and slowly varying responses, and to properly establish the design load effects. This poses some challenges, since wind cannot be considered stationary over time scales as long as 3 to 6 hours.

Guidance note:

Wind analyses are usually carried out as stationary analyses, so one option for circumventing the issue of nonstationarity over timespans in the order of 3 to 6 hours could be to carry out a number of stationary analyses over shorter timespans and combine them in a proper way.

Another option is to assume stationarity over the 3- to 6-hour timespans needed, regardless that this stationarity assumption is not fulfilled as far as the wind is concerned, and then combine with suitable conservative analysis assumptions. In general, when it can be demonstrated that considering stationary conditions over 3 to 6 hours yields less favourable loads, stationary wind conditions over such long periods can be assumed in design.

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

Simulations for determination of loads and responses involve combinations of wind and waves. Such combinations will require conversion of significant wave heights and mean wind speeds from their respective (usually different) reference periods to a common reference period equal to the chosen simulation length. Reference is made to conversion formulas for significant wave height in [3.6.1.3] and conversion formulas for mean wind speed in DNVGL-RP-C205. For a simulation length of 1 hour, it is recommended to apply the same conversion as the one recommended in IEC61400-3, Section 7.4.6.

4.6.2.2 A coupled analysis in the time domain shall be carried out for floating units in the operational phases for the wind turbine.

4.6.2.3 Software to be used in design shall be validated by model tests to show applicability for the specific design.

4.6.3 Design load cases

4.6.3.1 DNVGL-ST-0437, provides a number of proposed design load cases by combining various environmental conditions. This table shall be supplemented by the floater-specific design load cases proposed in Table 4-4. For all load cases given in DNVGL-ST-0437, including the ones where no current is specified, an applicable current model shall be included.

Table 4-4 Proposed floater-specific design load cases

Design situation	*DLC*	*Wind condition*	*Sea state condition*	*Wind and wave directionality*	*Currents*	*Water level*	*Other conditions*	*Load factor according to Table 5-1 (1)*
Power production + occurrence fault	2.6	NTM	NSS	Misaligned Multiple directions	NCM	MSL	Transient condition between intact and redundancy check condition	(e)
	2.7	NTM	NSS	Misaligned Multiple directions	NCM	MSL	Redundancy check condition	(e)
	2.8	NTM	NSS	Misaligned Multiple directions	NCM	MSL	Leakage (damage structure)	(e)

Design situation	*DLC*	*Wind condition*	*Sea state condition*	*Wind and wave directionality*	*Currents*	*Water level*	*Other conditions*	*Load factor according to* Table 5-1 *(1)*
Parked (2) + occurrence fault	7.3	EWM (turbulent) U_{hub} = U_{1year}	ESS $H_s = H_{s,1\text{-}yr}$	Misaligned Multiple directions	ECM (1 year)	EWLR (1 year)	Transient condition between intact and redundancy check condition	(e)
	7.4	EWM (turbulent) U_{hub} = U_{1year}	ESS $H_s = H_{s,1\text{-}yr}$	Misaligned Multiple directions	ECM (1 year)	EWLR (1 year)	Redundancy check condition	(e)
	7.5	EWM (turbulent) U_{hub} = U_{1year}	ESS $H_s = H_{s,1\text{-}yr}$	Misaligned Multiple directions	ECM (1 year)	EWLR (1 year)	Leakage (damage structure)	(e)
Normal shutdown	4.3	NTM $V_{in} < V_{hub} < V_{out}$	SSS or the most severe conditions that triggers the safety limits of the control	Misaligned Multiple directions	NCM	MSL	Maximum operating sea state limit (if present)	(a), (b)
(1) Special safety factors apply for mooring lines and anchors in the station keeping system, see Sec.8 and Sec.9, respectively. (2) Standing still or idling. (2) Standing still or idling.								

The design load cases denoted 2.6 and 7.3 refer to the transient condition between intact and damaged station keeping system.

The design load cases denoted 2.7. and 7.4 refer to the stationary situation after the loss of one mooring line or tendon. How the safety system detects the damage and the resulting action shall be considered.

The design load case denoted 2.8 and 7.5 refer to the situation where the structure is damaged. How the safety system detects the damage and the resulting action shall be considered.

The design load case denoted 4.3 correspond to a situation where the safety limits, e.g. acceleration, of the control system are triggered.

Metocean acronyms are explained in detail in DNVGL-ST-0437.

4.6.3.2 The proposed design load cases in Table 4-4 as well as in DNVGL-ST-0437 as referenced in [4.6.3.1] shall be supplemented as necessary with load cases accounting for:

— changes necessitated by new or revised representations of gust situations as addressed in Sec.3
— changes necessitated by floating wind turbines, in particular those reflecting that the control system is used to keep the turbine in place by minimizing excitation.

4.6.3.3 For ULS load cases it is important to assume the most unfavourable directions of the wind and the waves.

4.6.4 Ice loads

4.6.4.1 Proposed load cases for ice loading, given in DNVGL-ST-0437, shall be supplemented as necessary by supplementary load cases of importance for the mooring system, for example if locations in cold climates such as the Gulf of Bothnia are considered.

4.6.4.2 Ice loads can form the governing loads for the design of mooring systems, for example when an ice ridge or a large ice floe hits the hull of the floating unit. Such loading can be assessed by model testing.

4.6.5 Fatigue loads

4.6.5.1 The long term wind and wave environment can be represented by a number of discrete conditions. Each condition consists of a reference wind direction, a reference wave direction and a reference sea state characterized by a significant wave height, peak period, current velocity, mean wind speed, and standard deviation of wind speed. The probability of occurrence of each of these conditions must be specified. In general 8 to 12 reference directions provide a good representation of the directional distribution of a long-term wave environment. The necessary number of reference sea states can be in the range of 10 to 50. Fatigue damage prediction can be sensitive to the number of sea states in this discretization, and sensitivity studies are therefore necessary.

4.6.5.2 All significant stress ranges, which contribute to fatigue damage in the structure, shall be considered. Stress ranges caused by wind and wave loading shall normally be established by time domain analysis under due consideration of the motion characteristics of the floater and its interaction with the control system.

Stress ranges caused by wave and wind loading shall be established from site-specific environmental data. The actual alignment of the rotor axis of the wind turbine relative to the direction of the wind can be important.

Stress ranges caused by abnormal wind turbine loads shall be considered as appropriate. Stresses from persistent errors and from relatively frequently occurring faults might be relevant.

If frequency domain analysis is used, validation against model tests or time domain analysis shall be performed. Frequency domain analysis should only be applied in the preliminary design phase, not for final documentation of the FLS capacity.

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

4.7 Accidental loads (A)

4.7.1 Definition

4.7.1.1 Accidental loads are loads related to accidental events, abnormal operations or technical failure, i.e. events that occur more rarely than the 50- or 100-year loads usually used as characteristic loads for design in the ULS. Characteristic accidental loads are accordingly selected such that they have annual exceedance probabilities in the range 10^{-4} to 10^{-2}. Unless specified otherwise, characteristic accidental loads for floating wind turbine structures and their station keeping systems shall be taken as 500-year loads, i.e. loads with an annual probability of exceedance of $2 \cdot 10^{-3}$. For floating support structures, accidental loads are foreseen first of all to be loads due to

— impacts from unintended collisions by drifting service vessels
— unintended change in ballast distribution (e.g. failure of active ballast system)
— change of intended pressure difference
— loss of mooring line or tendon

— dropped objects
— fire and explosions
— accidental flooding.

4.7.1.2 Accidental loads from unintended collisions with drifting service vessels shall be considered and shall meet the requirements specified in [4.5.2].

4.7.1.3 Accidental loads from dropped objects shall be considered as they can be expected from normal maintenance operations where objects are lifted and at the risk of being dropped.

4.7.1.4 For structures which are to be designed for use in areas exposed to rare tropical storms, loads due to such tropical storms can be rationalized as accidental loads.

4.8 Deformation loads (D)

4.8.1 Definition

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

— temperature loads
— built-in deformations
— creep loads
— settlements of foundations.

Guidance note:

Creep loads can result from time-dependent elongation of fibre ropes. Creep loads are addressed in detail in DNVGL-OS-E303.

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4.8.1.2 Requirements for deformation loads are given in DNVGL-ST-0126.

4.9 Pressure loads on hull

4.9.1 General

4.9.1.1 Loads on hull structures consist of tank pressures and sea pressures and comprise pressure loads in the following categories:

— permanent loads from ballast water
— variable functional loads from ballast water
— permanent hydrostatic loads from seawater
— variable environmental loads from seawater.

4.9.1.2 For the design of tanks, i.e. both ballast tanks and other tanks in the hull, local tank loads are specified as tank pressures and sea pressures in [4.9.2] and [4.9.3], respectively. Typical combinations for local tank pressures and sea pressures are addressed in [4.9.4].

4.9.1.3 The design shall include the effects of relevant global and local responses. The tightness of the tank should be tested during construction. The testing conditions should, at a minimum, represent the maximum static pressure during operation. When testing is to be performed, requirements for structural tests as well as for testing for watertightness specified in DNVGL-OS-C401 may be applied.

4.9.1.4 For arrangements with free flooding or level alarms installed, limiting the operational tank pressures, it shall be ensured that the tank will not become overpressurized during operation and tank testing conditions.

4.9.2 Tank pressures

4.9.2.1 Tanks shall be designed for the maximum filling height. Three alternative tank filling types are defined as specified in [4.9.2.2], [4.9.2.3] and [4.9.2.4], respectively. Requirements for the design tank pressures for these three alternatives are given in [4.9.2.5]. Additional requirements for a fourth alternative, referring to tanks that can be emptied by air pressure, are specified in [4.9.2.6].

4.9.2.2 Alternative 1: Tank filling for units without internal tanks or watertight decks:

— water is filled up to the level assumed in the design analyses
— the water level should be easy to check during installation and operation
— automatic or manual routines for regular checking shall be in place.

4.9.2.3 Alternative 2: Tank filling for tanks with maximum filling height less than at the top of the air pipe:

a) filling by pumps with tank level alarms installed:
   — applicable for arrangements with limitations of the possible filling height
   — the tank is arranged with an alarm system installed to limit the maximum pressure height level
   — criteria applicable for the tank filling arrangements are given in DNVGL-OS-D101. Such arrangement should have a high level alarm and a high-high level alarm with automatic shut-off of the pump. Consequence of possible failure of the alarm system may be considered as an accidental event
   — the dynamic pressure head due to the operation of the pumps, p_{dyn}, may normally be neglected, provided the shut-off level is set to maximum 98% of the tank height.

b) filling by free flooding:
   — applicable for arrangements where the tanks are filled by gravity, without pumps
   — criteria applicable for the tank filling arrangements are given in DNVGL-OS-D101, for free flooding ballast systems
   — the dynamic pressure head due to the operation of the pumps, p_{dyn}, may be neglected.

4.9.2.4 Alternative 3: Tank filling for tanks with maximum filling height to the top of the air pipe:

— applicable for arrangements with no limitations of the possible filling height
— the tank is filled by pumps
— in addition to the static pressure head to the top of the air pipe, the dynamic pressure head due to flow through air pipes caused by the operation of the pumps, p_{dyn}, should be considered.

4.9.2.5 The design tank pressure acting on the internal tank wall shall be taken as:

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

in which:

a_v = maximum vertical acceleration (m/s^2), taken as the coupled motion response applicable to the tank in question. For preliminary design calculations of tank pressures, a_v may be taken as $0.25g_0$. For final design, a_v shall be documented, e.g. by dynamic analysis

g_0 = 9.81 m/s^2, acceleration of gravity

ρ = density of liquid, minimum density equal to that of seawater (1025 kg/m^3)

$\gamma_{f,G,Q}$ = load factor for permanent and variable functional loads, see Sec.5

$\gamma_{f,E}$ = load factor for environmental loads, see Sec.5

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

Parameters used in the expression for the design tank pressure are illustrated in Figure 4-1.

4.9.2.6 Alternative 4: For tanks that can be emptied by air pressure, the following additional requirements apply:

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

$$p_d = \left(\rho \cdot g_0 \cdot h_{op} + p_{dyn}\right) \cdot \gamma_{f,G,Q}$$

in which p_{dyn} = pressure due to flow through pipes, minimum 25 kN/m^2.

If the tank shall be emptied by air pressure, the pressure in the tank will be constant over the total height of the tank walls, as well as the top and bottom surfaces. The height h_{op} shall be calculated from the bottom of the considered tank, see Figure 4-1 Alt. 4b.

Guidance note:

The internal pressure specified in [4.9.2.6] need not be combined with extreme environmental loads. Normally only static global response need to be considered.

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

4.9.2.7 The top of the air pipe shall always be located above the water line.

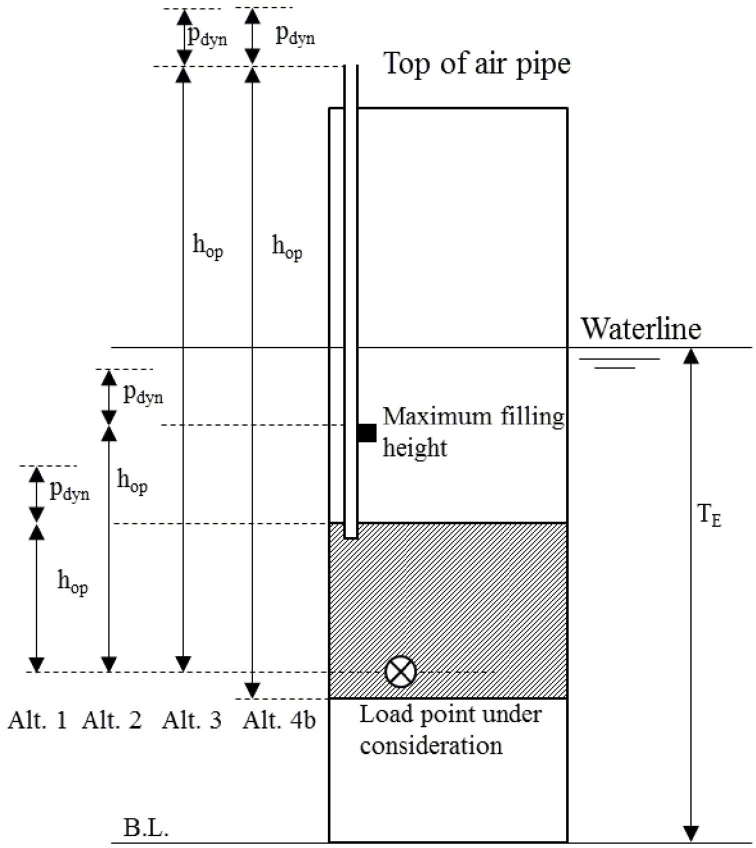

Figure 4-1 Parameters for internal tank pressures

4.9.3 Sea pressures for the ultimate limit state

4.9.3.1 The design sea pressure acting on slender floating wind turbine units in operating conditions in deep waters may, if not more refined analyses are performed, be taken as:

$$p_{d,ULS} = p_s \cdot \gamma_{f,G,Q} + p_e \cdot \gamma_{f,E}$$

in which:

p_s = $\rho\, g_0\, (T_E - z_b)$ (kN/m^2) ≥ 0

p_e = $\rho\, g_0\, (D_D - z_b)$ (kN/m^2) for $z_b \geq T_E$

= $0.5\rho\, g_0\, He-^{kz}\cos\theta$ for $z_b < T_E$

T_E = extreme operational draught (m) measured vertically from the moulded baseline (BL) to the assigned load waterline. The operational draught should be varied considering all relevant effects such as set-down, storm surge, tide, trim/heel angle and heave effects.

D_D	=	vertical distance (m) from the moulded baseline to the wave crest. Relative heave motions shall be considered.
z_b	=	vertical distance (m) from the moulded base line to the load point
p_s	=	static sea pressure
p_e	=	dynamic (environmental) sea pressure.
H	=	trough-to-crest wave height
θ	=	$kx-\omega t = k(x-ct)$
x	=	distance of propagation
c	=	wave celerity
ω	=	$2\pi/T$= angular wave frequency
k	=	$2\pi/\lambda$ = wave number, infinite water depth
z	=	$(T_E - z_b)$ = distance from mean free surface (defined as positive)
λ	=	wave length

For shallow water depths the design sea pressure can be calculated in accordance with DNVGL-RP-C205.

Requirements for load factors are given in Sec.5.

Parameters used in the expression for the design sea pressure are illustrated in Figure 4-2.

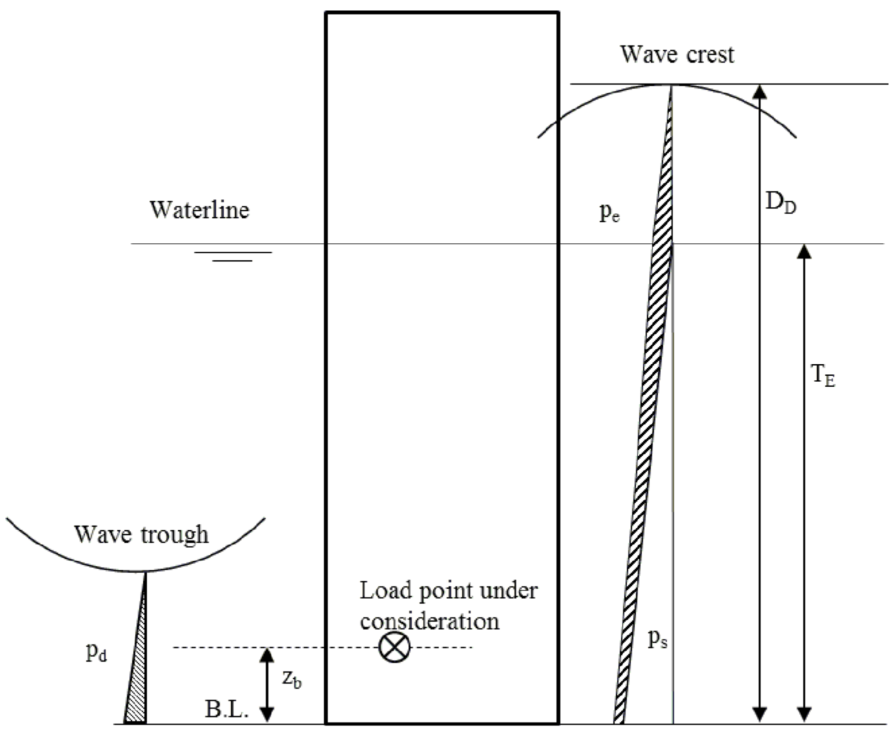

Figure 4-2 Parameters for sea pressures

4.9.4 Combination of tank pressures and sea pressures

Examples of considerations that should be evaluated in connection with the load cases of local pressures acting on the substructure of a floating wind turbine unit are given below:

— Relevant combinations of internal tank pressures and external sea pressures for tanks should be considered for both the intact and damaged load conditions.
— Maximum external pressure shall be combined with minimum internal pressure.
— Minimum external pressure shall be combined with maximum internal pressure.
— For tanks separated by internal watertight bulkhead/deck, the internal tank pressure should normally not be considered to act simultaneously on both sides of the bulkhead/deck. Combinations with maximum tank pressure from each of the tanks and zero tank pressure from the neighbouring tank should be considered if relevant. Effects of sea pressure response on the internal watertight bulkhead/deck should be assessed and included when relevant.
— For units with complex geometry local structural models should be created in order to evaluate responses of the structure to various sea and tank pressures. If cross-section arrangements change along the length of the structure, several local models may be required in order to fully evaluate local response at all relevant sections.
— The intention of the local model is to simulate the local structural response for the most unfavourable combination of relevant local loads. Loads are usually applied in the analysis models at the girder level and not at the individual stiffener level (typical global and in some cases local analysis models). In such cases the local stiffener bending is not included in the model responses. The stiffener bending response will then be explicitly included in the buckling code check as lateral pressure (for plate induced and/or stiffener induced buckling).

4.9.5 Superimposition of responses

4.9.5.1 The simultaneity of the responses resulting from the local and global analysis models, including various sea and tank pressures, may normally be accounted for by linear superposition of the responses for logical load combinations.

4.9.5.2 When evaluating responses by superimposing stresses resulting from several different models, consideration shall be given to the following:

— loads applied in global and local models
— relevant combination of tank and sea pressures
— it should be ensured that responses from design loads are not included more than once.

4.9.5.3 Further information regarding superimposition of loads from local and global models can be found in DNVGL-RP-C103 Sec.4.

4.9.6 Wave slamming

4.9.6.1 Parts of the structure that are located near the water surface are susceptible to forces caused by wave slamming when the structural parts penetrate the water surface. Slamming is due to sudden retardation of a volume of fluid. The retardation causes a considerable force to act on the structure. Wave slamming may also be caused by breaking waves, followed by drag and inertia forces.

4.9.6.2 Wave slamming may have both global and local effects. The impact of a massive bulk of water from a wave crest hitting a deck structure is a global load effect, while wave slamming on a brace is a local load effect which usually does not compromise the global structural capacity.

4.9.6.3 For further information regarding wave slamming and its representation, see DNVGL-RP-C205 and DNVGL-OTG-14.

SECTION 5 LOAD FACTORS AND MATERIAL FACTORS

5.1 Load factors

5.1.1 Load factors for the ultimate and accidental limit state

5.1.1.1 Table 5-1 provides five 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 analysis of the ULS for abnormal wind load cases as defined in DNVGL-ST-0437, usually associated with fault design situations, the set denoted (c) shall be used. For analysis of the ALS for intact structures subject to loads such as impact from a free-floating service vessel, the load factor set denoted (d) shall be used. For analysis of the ALS for damaged structures, such as after a collision with a free-floating service vessel ([4.5.2]), the load factor set (e) 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 floating support structures and their station keeping systems. The load factors for environmental loads depend on the consequence class required for the structural component in question. In special cases, such as for certain components of the station keeping system, other load factor requirements may apply and are then specifically stated together with the design rules in which they are used. In particular, for design of mooring lines and anchor foundations for mooring lines, separate load factor requirements are given and overrule load factor sets (b), (d) and (e) in Table 5-1, see Sec.8.

Table 5-1 Load factors γ_f for the ULS

Load factor set	*Limit state*	*Load categories*					
		G	*Q*	*E*		*D*	*P*
				Consequence class			
				1	*2*		
(a)	ULS	1.25	1.25	0.7 1)		1.0	0.9/1.1 3)
(b)	ULS	1.0 2)	1.0	1.35	1.55	1.0	0.9/1.1 3)
(c)	ULS for abnormal wind load cases as defined in DNVGL-ST-0437	1.1	1.1	1.1	1.25	1.0	0.9/1.1 3)
(d)	ALS for intact structure	1.0	1.0	1.0	1.15	1.0	0.9/1.1 3)
(e)	ALS for damaged structure	1.0	1.0	1.0	1.15	1.0	0.9/1.1 3)

Load factor set	Limit state	Load categories					
		G	Q	E		D	P
				Consequence class			
				1	2		

Load categories are:
G = permanent load
Q = variable functional load, normally relevant only for design against boat impacts and for local design of platforms
E = environmental load
D = deformation load
P = prestressing load
For description of load categories, see Sec.4
1) When environmental loads are to be combined with functional loads from boat impacts, the environmental load factor shall be increased from 0.7 to 1.0 to reflect that boat impacts are correlated with the wave conditions
(2) It is assumed that tight weight control of the structure is performed for floating structures. If sensitivity studies show risk for excessive dynamic excitations, the load factors for permanent loads shall be varied between 0.9 and 1.1
(3) The most conservative value of 0.9 and 1.1 shall be used as load factor the design

Guidance note:

Load factor set (a) is used for ULS design of primary and secondary structures when the permanent load or the variable functional load is the dominating load. Design against pretension, lifting forces and hydrostatic pressures forms examples where load factor set (a) is governing. Also, load factor set (a) is of relevance for design of secondary structures such as boat landings, fenders and lay down areas, for which variable functional loads from boat impacts are the dominating loads. Load factor set (b) is used for ULS design when the environmental load is the dominating load.

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 boat impact load from a service vessel with an environmental load in accordance with 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 results from 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---o-f---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 in accordance with Sec.4, and representing the load effect that results from two or more concurrently acting load processes.

5.1.2 Load factor for the fatigue limit state

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 γ_f in the FLS is 1.0 for all load categories.

5.1.3 Load factor for the serviceability limit state

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

5.2 Material factors

5.2.1 Material factors for the ultimate limit state

Material factors for the ULS are given in the relevant sections for design in the ULS. These material factors apply to design of support structures and anchor foundations.

5.2.2 Material factors for the fatigue limit state

Material factors as such are not specified for the FLS. Design against the FLS is based on a format which makes use of an overall design fatigue factor (DFF) applied to a characteristic cumulative damage. Requirements for design fatigue factors for the FLS are given in the relevant sections for design in the FLS.

5.2.3 Material factors for the accidental limit state and the serviceability limit state

Unless otherwise stated, the material factor γ_m for the ALS and the SLS shall be taken as 1.0.

SECTION 6 MATERIALS

6.1 Introduction

6.1.1 General

6.1.1.1 Material specifications shall be established for all structural materials utilized in a floating wind turbine structure and its station keeping system. Such materials shall be suitable for their intended purpose and have adequate properties in all relevant design conditions. Material selection shall be undertaken in accordance with the principles given in DNVGL-ST-0126.

6.1.1.2 When considering criteria appropriate to material grade selection, adequate consideration shall be given to all relevant phases in the life cycle of the unit. In this context there may be conditions and criteria, other than those from the in-service, operational phase, which may govern the design requirements with respect to the selection of materials. Such criteria may, for example, consist of design temperature and stress levels during marine operations.

6.1.1.3 Material designations are defined in DNVGL-ST-0126.

6.2 Selection of metallic materials

6.2.1 General

6.2.1.1 For selection of structural steel for design and construction of floating support structures, reference is made to the materials section of DNVGL-ST-0126.

6.2.1.2 For selection of steel to be used in station keeping systems, such as steel for tendons and steel for chains used in mooring lines, reference is made to [6.6].

6.2.2 Structural steel and aluminium

6.2.2.1 Special material requirements to support structural integrity are given in [6.2.2.2] to [6.2.2.5].

6.2.2.2 Materials for:

— rolled steel for structural applications and pressure vessels
— steel tubes, pipes and fittings
— steel forgings
— steel castings.

shall comply with the requirements set forth in DNVGL-ST-0126.

6.2.2.3 In structural cross-joints essential for the overall structural integrity where high tensile stresses are acting normal to the plane of a plate, the plate material shall be tested to prove the ability to resist lamellar tearing (Z-quality).

6.2.2.4 Stainless steel shall have a maximum carbon content of 0.05%. The stainless steel material shall be in the white pickled and passivated condition.

6.2.2.5 Aluminium shall be of seawater resistant type. Aluminium alloys shall comply with the requirements set forth in DNVGL-OS-B101. Aluminium shall not be used as construction material in load-bearing structures.

6.2.3 Bolting materials

6.2.3.1 Bolting materials for structural applications shall in general be carbon steels or low-alloy steels with the limitation that the hardness and strength class shall not exceed ISO 898 Class 8.8. For bolting materials to be used in towers, the hardness and strength class shall not exceed ISO 898 Class 10.9.

6.2.4 Structural category

6.2.4.1 The purpose of structural categorization is to assure adequate material quality and suitable inspection to avoid brittle fracture. Part of the purpose of inspection is to detect defects that may grow into fatigue cracks during the service life.

6.2.4.2 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 which in combination with possible weld defects or fatigue cracks may provoke brittle fracture.

The structural category for selection of materials shall be determined in accordance with principles specified in Table 6-1, see also DNVGL-ST-0126.

Table 6-1 Structural categories for selection of materials

Structural category	*Principles for determination of structural category*
Special	Structural parts where failure will have substantial consequences and are subject to a complex stress condition. Structural parts with conditions that may increase the probability of a brittle fracture.[1)]
Primary	Structural parts where failure will have substantial consequences.
Secondary	Structural parts where failure will be without significant consequence.
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.	

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

6.2.4.4 Categorization of hull structures shall adhere to the principles given in [6.2.4.2]. Examples of implementation of these principles for typical floating hull structures can be found in DNVGL-OS-C103 for column-stabilized units, DNVGL-OS-C105 for TLPs and DNVGL-OS-C106 for spar structures. For tubular structures reference is made to DNVGL-ST-0126.

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

6.3 Selection of concrete materials

For selection of concrete materials for design and construction of floating support structures, reference is made to the materials section of DNVGL-ST-0126. Useful information for selection of concrete materials can also be found in DNVGL-ST-C502.

6.4 Selection of grout materials

For selection of grout materials for design and construction of grouted connections in floating support structures, reference is made to DNVGL-ST-0126.

6.5 Selection of composite materials

Owing to their favourable weight properties, composite materials such as fibre-reinforced plastic laminates and sandwich constructions may prove attractive for construction of selected structural components in floating wind turbine structures, such as towers. For selection of composite materials, reference is made to DNVGL-ST-C501.

6.6 Selection of materials for chains, wires and tethers

6.6.1 General

6.6.1.1 When designing a mooring system it is of vital importance that the components of the individual mooring lines (or tendons) match one another, i.e. no adverse effects such as corrosion shall be introduced from one component to the next.

6.6.1.2 Due consideration shall be given to fatigue loads generated by a taut fibre rope, and how this affects chain and wire-rope fatigue life. On this basis, it is generally recommended to use lower-grade steel materials, in particular for chain loaded in a configuration with little or no catenary.

6.6.1.3 The torque and twist characteristics should be matched between all elements of each mooring line or tendon. Any undue twisting of steel-wire rope will affect fatigue life. Chain should only be connected to torque-neutral fibre ropes.

6.6.1.4 Requirements for system analysis are found in DNVGL-OS-E301.

6.6.2 Chains

6.6.2.1 Steels considered for rolled steel bars, steel forgings and steel castings to be used in the manufacture of offshore mooring chain and accessories are classified by specified minimum ultimate tensile strength into six grades: R3, R3S, R4, R4S, R5 and R6. Specific requirements for each grade are given in DNVGL-OS-E302.

6.6.2.2 Recommended values for corrosion allowance, defined as an addition to the chain diameter, are given in Sec.13.

6.6.3 Steel wires

6.6.3.1 The lifetime of a steel wire rope is dependent on the layout and design of the wire and on the degree of protection. Guidance for selection of the type of steel wire rope depending on the specified design life is given in Table 6-2.

Table 6-2 Choice of type of steel wire rope

Design life (years)	*Possibility for replacement of wire rope segments*	
	Yes	*No*
< 8	A/B/C	A/B/C
8 to 15	A/B/C	A/B
> 15	A/B	A
A) Half locked coil/full locked coil/spiral rope with plastic sheathing B) Half locked coil/full locked coil/spiral rope without plastic sheathing C) Stranded rope		

6.6.3.2 Details regarding steel wire ropes for mooring lines can be found in DNVGL-OS-E304.

6.6.4 Fibre ropes

6.6.4.1 Design of fibre rope systems for mooring of floating wind turbine structures should be performed as an optimization process between selection of change-in-length performance and 3-T loadbearing capacity. The specific change-in-length characteristics are determined in the selection of loadbearing material, and then the balance between change-in-length characteristics for the rope and the 3-T capacity for the rope is determined by choosing the appropriate nominal loadbearing linear density. The change-in-length performance is proportional to the original length of the rope.

6.6.4.2 Requirements for offshore fibre ropes and tethers are given in DNVGL-OS-E303. Recommendations are provided in DNVGL-RP-E304 for condition management and in DNVGL-RP-E305 for design, testing and analysis.

6.7 Selection of materials for electrical cables

6.7.1 General

6.7.1.1 Requirements for material selection for electrical cables are given ISO 13628-5 shall apply.

6.7.1.2 Useful guidance for material selection for electrical cables is given in EEMUA publication no. 194.

6.8 Selection of materials for solid ballast

The type and use of permanent ballast for stability purposes, for example within the internal compartment of a spar unit, must be carefully evaluated with respect to long term effects such as those related to corrosion and washout. When selecting solid ballast material, caution must be exercised to avoid materials such as contractant sands which are susceptible to liquefaction and which may shift location in the liquefied condition and compromise the stability.

Guidance note:

Decommissioning or methods for removal may be considered in the choice of grain size of solid ballast.

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SECTION 7 STRUCTURAL DESIGN

7.1 Introduction

7.1.1 General

7.1.1.1 This section applies to structural design of hull, mooring lines, tendons and anchors in floating wind turbine structures and their station keeping systems. The requirements for structural design given in DNVGL-ST-0126 apply to floating wind turbine structures with the exceptions, deviations and additional requirements specified in this section. In particular, the requirements for material factors specified in DNVGL-ST-0126 apply unless otherwise specified in this section.

7.1.1.2 Determination of hydrodynamic loading and response is vital for structural design, since this loading and response involve wave excitation, added mass, wave and viscous damping, stiffness as well as the geometry of the floating structure. All these parameters are decisive for determination of wave frequency (WF), low frequency (LF) and high frequency (HF) floater motions. All these motion components are of importance and must be determined carefully. For further details, reference is made to DNVGL-RP-C205 and DNVGL-RP-F205 Sec.7.

7.1.1.3 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.

7.1.1.4 Structural steel elements shall be manufactured in accordance with the requirements given in DNVGL-OS-C401.

7.1.2 Interface with wind turbine

7.1.2.1 The rotating turbine will influence the global motions of the floating wind turbine structure in the operating mode. A control system shall be in place for the floater as required in Sec.11. The control system may include specific software and algorithms developed based on a combination of model tests and advanced analyses, and this control software and the algorithms may be used during operation to limit the floater motions. The roll and pitch wind damping effects may be vital in relation to reducing the inclinations and thereby reducing the motions and accelerations as well as the global bending moments in the substructure and the tower. Such control software and algorithms will also have effects on mooring and cable hang-off motions for fatigue considerations. The coupling between the yaw and pitch modes of motion may be important for floaters with low yaw resistance, such as spars, since a tilted rotor will result in yaw loads.

7.1.2.2 The aero-hydro coupling of the wind turbine with the supporting floating structure shall be investigated in detail. Any software used for this purpose shall be validated against other validated software or against model tests or full scale tests.

7.1.2.3 A sufficient air gap between the lowest blade tip position of the wind turbine and the sea surface 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 blades. The evaluation of the air gap shall be carried out with due consideration of the wave process and the floater motions. When the air gap has been calculated it is recommended to consider an extra allowance of at least 1.0 m.

Guidance note:

DNVGL-RP-C205 and DNVGL-OTG-13 provide useful guidance for air gap analysis.

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7.1.3 Design against undesirable effects

7.1.3.1 It shall be ensured that effects like Mathieu Instability (MI) and vortex induced motions (VIM) are considered in the design. Selection of natural periods in heave, roll and pitch for a DDF is one example where caution in this respect must be exercised. Another case where caution must be exercised with respect to Mathieu Instability refers to floater concepts with abrupt changes in water-plane stiffness and metacentric height. Model testing forms an excellent method to verify that effects like MI and VIM are unlikely to occur or are controllable, but state-of-the-art offshore design practices and methodologies can provide sufficient guidance in early design stages. The effect of the turbine controller shall be considered when effects like MI and VIM are addressed.

Guidance note:

DNVGL-OS-C106 and DNVGL-RP-F205 Sec.7 provide useful information regarding VIM. MI may typically occur as the result of large heave motion if the natural period of heave comes close or equal to half the natural period of pitch or half the natural period of roll.

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

7.1.4 Installation-friendly design

It shall be assessed which initiatives and actions can be taken in design in order to facilitate the installation and maintenance of the structure.

Guidance note:

The logistics for installation of many structures in a wind farm is a key issue, e.g. with a view to avoiding waiting times which are costly. Examples of issues to consider in this context are number and locations of assembly sites, transportation methods (transportation in floating condition vs. transportation on a vessel), and minimum draughts for different operations during the installation.

Even though the farm consists of many mass produced units, which in principle are identical, there will be individual adaptations, tailored for specific wind turbine positions and thereby posing challenges for an efficient and cost-effective installation on site. Navigation lights form one example of equipment which is individually tailored for each wind turbine position and implies an individual adaptation of the wind turbine structure in that position. Variable draught and bottom conditions form other examples that imply such individual adaptation in each individual wind turbine position.

It will in general contribute to an efficient installation to consider constructability and installability of the wind turbine structure when choosing the solution for the structure and carrying out the structural design of it.

It may in some cases contribute to an efficient installation if the weather criterion for installation of the support structures and mooring lines as well as wind turbine and cables is considered in the design to ensure that no bottle-necks occur in consecutive installation operations due to weather sensitivity.

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

7.1.5 Special provisions for global analysis

7.1.5.1 P-delta effects due to floater heel or trim shall be documented by a global analysis, as they can be expected to be significant due to large rotor and nacelle weights. Global bending forces and shear forces along the height of the tower and substructure due to environmental load effects shall be determined for the ULS and the FLS as well as for the ALS.

7.1.5.2 Loads which are associated with the particular type of floating structure shall be considered, for example

— splitting forces for semi-submersibles
— global bending of spar buoys
— bending and shear of ship-shaped structures
— ringing of slender hulls under hydrodynamic and aerodynamic loads.

7.1.5.3 Analytical models shall adequately describe the relevant properties of masses, loads, load effects, stiffness, motions and displacements, and shall satisfactorily account for local effects and system effects of time dependency, damping, drag loads and inertia loads. Hybrid analytical models may be necessary in order to capture both drag and inertia load and damping reasonably accurately.

7.1.6 Special provisions for plating and stiffeners in steel structures

7.1.6.1 The requirements in [7.1.6.2] to [7.1.6.7] will normally give minimum scantlings to plate and stiffened panels with respect to yield.

7.1.6.2 The buckling stability of plates shall be checked in accordance with DNV-RP-C201.

7.1.6.3 The thickness of plates shall not be less than:

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

f_{yd} = design yield strength f_y/γ_m
f_y is the minimum yield stress (N/mm^2)
t_0 = 7 mm for primary structural elements
5 mm for secondary structural elements
γ_m = material factor for steel
1.10.

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

$$t = \frac{15.8 \cdot k_a \cdot k_f \cdot s \cdot \sqrt{p_d}}{\sqrt{\sigma_{pd1} \cdot k_{pp}}} \ (mm)$$

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_f = 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
σ_{pd1} = design bending stress
$1.3 \cdot (f_{yd} - \sigma_{jd})$, but less than $f_{yd} = f_y/\gamma_m$
σ_{jd} = equivalent design stress for global in-plane membrane stress
k_{pp} = fixation parameter for plate
1.0 for clamped edges

0.5 for simply supported edges.

Guidance note:

The design bending stress σ_{pd1} is given as a bilinear capacity curve.

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

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

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

l = stiffener span (m)
k_m = bending moment factor, see Table 7-1
σ_{pd2} = design bending stress
$f_{yd} - \sigma_{jd}$
k_{ps} = fixation parameter for stiffeners
1.0 if at least one end is clamped
0.9 if both ends are simply supported.

7.1.6.6 The formula given in [7.1.6.5] 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 [7.1.6.5] is multiplied by the factor

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

where

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

7.1.6.7 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:

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

In such cases the section modulus of the stiffener calculated as indicated in [7.1.6.5] is normally to be based on the following parameter values:

k_m = 8
k_{ps} = 0.9

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

Guidance note:

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

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7.1.7 Special provisions for girders and girder systems in steel structures

7.1.7.1 The requirements in [7.1.7.2] to [7.1.7.18] give minimum scantlings to simple girders with respect to yield. Further procedures for the calculations of complex girder systems are indicated.

7.1.7.2 The buckling stability of girders shall be checked in accordance with DNV-RP-C201.

7.1.7.3 The thickness of web and flange plating is not to be less than specified in [7.1.6.3] and [7.1.6.4].

7.1.7.4 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.

7.1.7.5 When boundary conditions for individual girders are not predictable due to dependence of adjacent structures, direct calculations in accordance with the procedures given in [7.1.7.13] to [7.1.7.18] will be required.

7.1.7.6 The section modulus and web area of the girder shall be taken in accordance with particulars as given in [7.1.7.9] to [7.1.7.12] for simple girders or [7.1.7.13] to [7.1.7.18] for complex girders. Structural modelling in connection with direct stress analysis shall be based on the same particulars when applicable

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

$$b_e = C_e \cdot b$$

where:

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 [7.1.7.10]
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 [7.1.7.10].

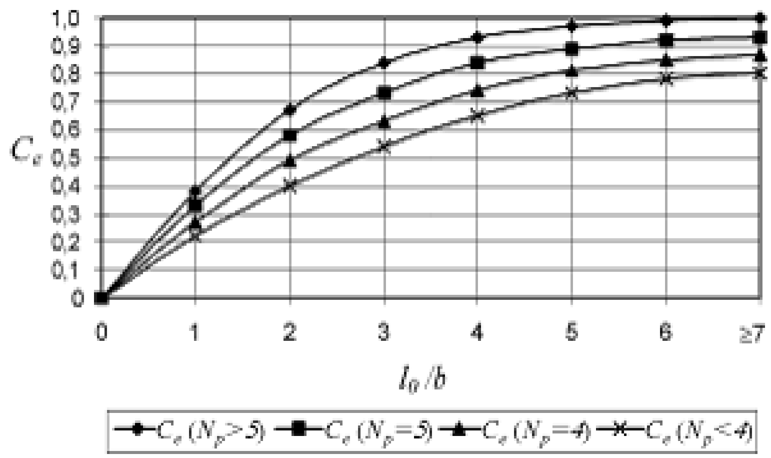


Figure 7-1 Graphs for the effective flange parameter C_e (from DNVGL-OS-C101)

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

7.1.7.9 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 in accordance with [7.1.7.10]
— net web area in accordance with [7.1.7.11].

7.1.7.10 Section modulus:

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

where:

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), may be determined as:
		$0.5 \cdot (l_1 + l_2)$ (m), l_1 and l_2 are the spans of the supported stiffeners, or distance between girders
k_m	=	bending moment factor k_m values in accordance with Table 7-1 may be applied
σ_{pd2}	=	design bending stress
	=	$f_{yd} - \sigma_{jd}$
σ_{jd}	=	equivalent design stress for global in-plane membrane stress.

7.1.7.11 Net web area:

$$A_W = \frac{k_\tau \cdot S \cdot b \cdot p_d - N_S \cdot P_{pd}}{\tau_p} \cdot 10^3 \ (mm^2)$$

k_τ = shear force factor k_τ may be in accordance with [7.1.7.12]

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

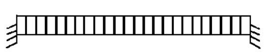
N_p = number of supported stiffeners on the girder span

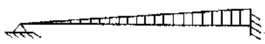
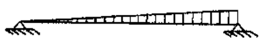
P_{pd} = average design point load (kN) from stiffeners between considered section and nearest support

τ_p = $0.5 \cdot f_{yd}$ (N/mm^2).

7.1.7.12 The k_m and k_τ values referred to in [7.1.7.10] and [7.1.7.11] may be calculated in accordance with general beam theory. In Table 7-1, k_m and k_τ values are given for some specified load and boundary conditions. Note that the smallest k_m value shall be applied to simple girders. For girders where brackets are fitted or the flange area has been partly increased due to large bending moment, a larger k_m value may be used outside the strengthened region.

Table 7-1 Values of k_m and k_τ (from DNVGL-OS-C101)

Load and boundary conditions			*Bending moment and shear force factors*		
Positions			1 k_{m1} $k_{\tau1}$	2 k_{m2} -	3 k_{m3} $k_{\tau3}$
1 Support	2 Field	3 Support			
			12 0.5	24	12 0.5
			0.38	14.2	8 0.63
			0.5	8	0.5
			15 0.3	23.3	10 0.7

Load and boundary conditions	*Bending moment and shear force factors*		
	0.2	16.8	7.5 0.8
	0.33	7.8	0.67

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

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

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

7.1.7.16 For systems consisting of slender girders, calculations based on beam theory (frame work analysis) may be applied. Analyses, where the stiffened plates are modelled with shell elements and the girders with beam elements, may also be applied. The beam elements should then be modelled without eccentricity relative to the plate to allow for derivation of the beam forces directly from the analysis. Due attention shall be paid to the following issues:

— shear area variations, e.g. cut-outs for stiffeners
— moment of inertia variation
— effective flange
— lateral buckling of girder flanges
— Lateral torsional buckling between tripping brackets.

7.1.7.17 The most unfavourable of the load conditions specified in Sec.4 shall be applied.

7.1.7.18 For girders taking part in the overall strength of the unit, stresses due to the design pressures specified in Sec.4 shall be combined with relevant overall stresses.

7.1.8 Special provisions for towers and access systems

7.1.8.1 The eigenfrequencies of the tower shall be calculated based on a structural model considering the entire floater. It is important that this eigenfrequency then is compared to the critical ranges of turbine operation (1P, 3P, etc.). It is important that the eigenfrequencies are adequately simulated in the global model used for design analyses, see also the guidance note in [1.1].

7.1.8.2 The tower access system consists of secondary structures such as boat landings, ladders and platforms. Requirements for such secondary structures are given in DNVGL-ST-0126.

7.1.8.3 Wave run-up shall be taken into account when secondary structures such as boat landings and access platforms attached to the floating structure are designed. This shall be done either by adequately increasing the air gap of the secondary structure to provide sufficient clearance against run-up in addition to the clearance against wave crests, or by designing the secondary structure against wave run-up loads.

The design of secondary structures attached to large body structures and exposed to wave run-up is often governed by the run-up.

Guidance note:

Wave run-up is a function of the wave steepness and the diameter of the large body structure in question and can be analysed by means of CFD. It is not feasible to perform long time simulations to obtain characteristic values of loads for secondary structures. Instead, a few trial design wave events, which are not necessarily regular waves, should be selected.

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7.2 Ultimate limit states for steel structures

7.2.1 General

Provisions and requirements for design of various types of structures and structural components against ultimate limit states (ULS) for normal loads are given in [7.2.2] through [7.2.6]. For capacity checks for accidental loads and in damaged conditions, see [7.4].

7.2.2 Ultimate limit state – tubular members, tubular joints and conical transitions

Tubular members, tubular joints and conical transitions shall be designed in accordance with NORSOK N-004. For structural members which are not exposed to external water pressure, other standards than NORSOK N-004 may be applied for design, e.g. Eurocode.

Guidance note:

Tubular members are members whose diameter-to-thickness ratio satisfies the following criterion:

$D/t \leq 120$, in which D = diameter and t = wall thickness.

Members with $D/t > 120$ are considered shell structures.

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7.2.3 Ultimate limit state – shell structures

7.2.3.1 Except for towers, shell structures shall be designed against buckling in accordance with DNVGL-RP-C202. For this purpose, the material factors specified in Table 7-2 apply. Shell structures in towers shall be designed against buckling in accordance with DNVGL-ST-0126.

Table 7-2 Material factors γ_m for buckling

Type of structure	*$\bar{\lambda} \leq 0.5$*	*$0.5 \leq \bar{\lambda} \leq 1.0$*	*$\bar{\lambda} \geq 1.0$*
Girder, beams stiffeners on shells	1.10	1.10	1.10
Shells of single curvature (cylindrical shells, conical shells)	1.10	$0.80 + 0.60\,\ell$	1.40

Guidance note:

Note that the slenderness is based on the buckling mode under consideration.

l = reduced slenderness parameter

$$= \sqrt{\frac{f_y}{\sigma_e}}$$

f_y = specified minimum yield stress

σ_e = elastic buckling stress for the buckling mode under consideration

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

7.2.3.2 In case solid ballast is used, the beneficial effect of horizontal pressure set up by the solid ballast and counteracting external pressure shall normally not be accounted for in the buckling checks for vertical shell elements. However, the beneficial effect of horizontal pressure from ballast water may always be accounted for in these checks. In the case that a horizontal earth pressure from saturated solid ballast can be documented, such an earth pressure may also be accounted for in the buckling checks. Finally, if the spaces between the solid ballast units are filled with concrete, cast in situ, this is regarded as a satisfactory stabilization of the outer shell.

Guidance note:

The issue here is that it is unclear whether earth pressures from saturated solid ballast against the shell elements will become mobilized and can be counted on. Such an earth pressure mobilization requires that the solid ballast becomes properly installed and distributed in the compartment in question to actually make contact with the compartment walls.

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7.2.4 Ultimate limit state – non-tubular beams, columns and frames

7.2.4.1 The design of non-tubular beams, columns and frames shall take into account the possible limits on the resistance of the cross section due to local buckling.

7.2.4.2 Buckling checks may be performed in accordance with EN 1993-1-1 for Class 1, 2 or 3 cross sections.

7.2.4.3 Capacity checks may be performed in accordance with EN 1993-1-1 for Class 1, 2 or 3 cross sections.

7.2.4.4 The material factors in accordance with Table 7-3 shall be used when EN 1993-1-1 is used for calculation of structural resistance.

Table 7-3 Material factors used with EN 1993-1-1

Type of calculation	*Material factor* [1)]	*Value*
Resistance of Class 1, 2 or 3 cross sections	γ_{M0}	1.10
Resistance of Class 4 cross sections	γ_{M1}	1.10
Resistance of members to buckling	γ_{M1}	1.10
1) Symbols in accordance with EN 1993-1-1.		

7.2.5 Ultimate limit state – special provisions for structural design of anchors

7.2.5.1 The design force, T_d, acting on the anchor and arising from line tension in a mooring line, which is hooked up to the anchor, shall be taken as equal to the design line tension in the mooring at the interface between the mooring line and the anchor, as resulting from calculations in accordance with specifications given in [7.4.2.2]. When more than one mooring line is hooked up to the anchor, the design force T_d, acting on the anchor, shall be calculated with due consideration of design force contributions from all mooring lines, one purpose being to avoid uplift and extraction of the anchor.

7.2.5.2 The design force, T_d, acting on the anchor and arising from tension in a tendon, which is hooked up to the anchor, shall be taken equal to the design tension in the tendon at the interface between the tendon and the anchor, as resulting from calculations in accordance with specifications of characteristic loads given in Sec.4 and requirements for load factors and load combinations given in [5.1.1]. When more than one tendon is hooked up to the anchor, the total design force T_d, acting on the anchor, shall be calculated with due consideration of design force contributions from all tendons.

7.2.5.3 Caution shall be exercised when the design force is established for an anchor which is used as a shared anchor point for anchoring of multiple wind turbine units, see [9.1.2.3] and [9.1.2.4].

7.2.6 Ultimate limit state – bolted connections

7.2.6.1 All bolts in the main load-bearing system of the primary structure shall be prestressed and the bolts shall satisfy the material requirements specified in Sec.6.

7.2.6.2 Slip-resistant bolt connections shall be designed in accordance with EN 1993-1-8.

7.2.6.3 End plate bolt connections shall be designed in accordance with EN 1993-1-8.

7.3 Fatigue limit states for steel structures

7.3.1 General

7.3.1.1 To ensure that the structure will fulfil its intended function, a fatigue assessment shall be carried out for each individual member, which is subjected to fatigue loading. Wherever appropriate, the fatigue assessment shall be supported by a detailed fatigue analysis.

Guidance note:

A fatigue assessment comprises a fatigue analysis as well as a capacity check.

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7.3.1.2 The design fatigue life for structural components shall be based on the specified service life of the structure. 20 years should normally be used as a minimum, regardless of whether the true service life is less than 20 years or not.

7.3.1.3 Complex connections of plated structures shall be documented in accordance with the hot spot methodology described in DNVGL-RP-C203.

7.3.1.4 Characteristic S-N curves for use in design against fatigue failure are given in DNVGL-RP-C203 Sec.7. The selection of the characteristic S-N curve for design of a particular structural detail shall be based on a classification of structural details depending on the corrosion protection of the structural surface and the location zone of the structural detail, i.e. whether the detail is in air, in seawater with adequate cathodic protection or in free corrosion conditions, see Table 7-4.

Table 7-4 Criteria for selection of characteristic S-N curves

Typical areas (structural detail categories)	*S-N curve designation*
Coated external areas above splash zone [1)] Internal coated area in dry condition Uncoated void tanks with a controlled non-corrosive environment	In air
Internal and external surface in splash zone above MWL [1)]	In air or Free corrosion [3) 4)]
Splash zone below MWL [1) 2)]	In seawater with cathodic protection
External surface below splash zone [1) 2)] Ballast tank below mean ballast water line [2)]	
Scour zone [1) 2)]	
Below scour zone [2)]	

1) Splash zone definition in accordance with Sec.13. If the splash zone can be inspected and repaired in dry and clean conditions, requirements for corrosion allowance and S-N curve can be waived. This is applicable for external and internal surfaces.
2) External surfaces below the MWL as well as ballast tanks, including those with a combination of solid and water ballast, shall be equipped with cathodic protection.
3) The basic S-N curve for unprotected steel in the splash zone is the curve marked free corrosion. The basic S-N curve for coated steel is the curve marked in air. It is acceptable to carry out fatigue life calculations in the splash zone based on accumulated damage for steel considering the probable coating conditions throughout the design life – intact, damaged and repaired. The coating conditions shall refer to an inspection and repair plan as specified in DNVGL-ST-0126.
4) When free corrosion S-N curves are applied in design, the full benefit of potential grinding of welds as outlined in [3.1.14] cannot be expected and therefore may not be taken into account.

The basis for the use of the S-N curves given in DNVGL-RP-C203 Sec.7 is that the fabrication of primary and special structures is in accordance with DNVGL-OS-C401. Special attention shall be paid to tolerances, welding quality and NDT. The weld connection between two components shall be assigned an inspection category in accordance with the highest category of the joined components.

7.3.1.5 Corrosion shall be taken into consideration where relevant. Both the thickness reduction from corrosion and the effect of the corrosion on the S-N curve shall be considered. Depending on the corrosion protection over the service life, different strategies for fatigue analyses are available:

1) The structure is protected against corrosion for the whole service life: no reduction in thickness and no change in S-N curve
2) The steel is coated with high quality coating: the coating is not maintained: gross scantlings to be used in global analyses, change in S-N curve at the end of the effective coating life to “free corrosion”, and corrosion allowance to be used for the remaining service life.
3) No coating and without effective cathodic protection: 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. S-N curves for free corrosion shall be used over the whole service life.

Strategy 3) is used for mooring chain, but is not a recommended strategy for structural steel, because it will require thickness measurements throughout the service life to document structural integrity and because fatigue cracks are in practice not possible to detect before they become of significant size.

For primary steel structures in the splash zone, which is defined in Sec.13, the corrosion allowance can be calculated from the corrosion rates specified in DNVGL-RP-0416 Sec.7. In the splash zone, the aim should be for use of high-performance coating in conjunction with reduced corrosion allowance. The coating should preferably be light-coloured to facilitate inspection and detection of cracks.

The additional costs of corrosion allowance for replaceable secondary structures should be balanced against the costs of replacement. For secondary structural components, the need for corrosion allowance may be assessed on an individual basis, for example considering a possible replacement of components.

7.3.1.6 Calculation of the fatigue life may be based on a fracture mechanics design analysis, either separately or as a supplement to an S-N fatigue calculation, see DNVGL-RP-C203. An alternative method for fracture mechanics analysis can be found in BS 7910.

7.3.1.7 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 local and global geometry. SCFs can be calculated based on information in relevant literature or by finite element analysis.

Guidance note:

In wind farms, where the same joint or structural detail is repeated many times in many identical support structures, requirements for cost-effectiveness make it particularly important to assess the SCFs accurately, and assessment by finite element analysis is recommended.

Relevant stress concentration factors can be found in DNVGL-RP-C203 Sec.7 and DNVGL-CG-0129.

Fabrication tolerances in accordance with category special in DNVGL-OS-C401 may be used to minimize the stress concentration factor for butt welds in cases where special efforts are paid to control the fabrication process.

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7.3.1.8 For fatigue analysis of base material, the stress ranges may be reduced prior to the fatigue analysis depending on the mean stress. For details regarding calculation of the acceptable stress range reductions as a function of the mean stress, reference is made to DNVGL-RP-C203.

7.3.1.9 Predictions of fatigue life may be based on calculations of cumulative fatigue damage under the assumption of linearly cumulative damage. 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 sum is used for prediction of linearly 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

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

D_C = characteristic cumulative damage

I = number of stress range blocks in a sufficiently fine, chosen discretization 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, e.g. obtained by rain-flow counting

$N_{C,i}$ = number of cycles to failure at the stress range $\Delta\sigma_i$ of the i^{th} stress block, interpreted from the characteristic S-N curve

7.3.1.10 The design cumulative damage D_D is obtained by multiplying the characteristic cumulative damage D_C by the design fatigue factor DFF

$$D_D = DFF \cdot D_C$$

7.3.1.11 Minimum requirements for the design fatigue factor DFF are given in Table 7-5. The design fatigue factors in Table 7-5 are based on the following assumptions:

— The floating structure is unmanned.
— The floating structure is on the designated site for the entire service life.
— The floating structure may have an inspection draught different from the operating draught.

7.3.1.12 The minimum design fatigue factors in Table 7-5 depend on the location of the structural detail and of the accessibility for inspection and repair. The design fatigue factors specified for structural details which are accessible for inspection are given with the prerequisite that inspections are carried out at intervals of four to five years. The relation between the level of inspection and the requirement for DFF is detailed in DNVGL-ST-0126. All surfaces designed to be inspection-free, regardless of whether they are accessible for inspection and repair, shall in design be treated as inaccessible.

Guidance note:

By proper planning, it may be possible to design underwater hulls in such a manner that critical welds are located on the inside of the hull and with access for inspection and repair.

Note that special provisions for design fatigue factors apply to mooring chain and steel tendons, see [8.2.5.1] and [7.6.2.10] , respectively, such that the design fatigue factors in Table 7-5 in practice apply to tower, floating substructure and anchors.

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Table 7-5 Minimum requirements for design fatigue factors, DFF

Structural element	*Typical structural components and areas*	*DFF*
(a) Internal structure, accessible and not welded directly to the submerged part.	Frames, bulkheads, stringers in voids, ballast tanks and tower.	1
(b) External structure, accessible for regular inspection and repair in dry and clean conditions.	Tower and substructure above lowest inspection or maintenance draught.	1
(c) Internal structure, accessible and welded directly to the submerged part.	Stiffeners, frames, bulkheads welded to external shell plate below highest loaded draught in voids, ballast tanks and tower.	2
(d) External structure not accessible for inspection and repair in dry and clean conditions.[1)]	External shell plate below lowest inspection or maintenance draught, bilge keel, fairlead structure.	2
(e) Non-accessible areas, areas not planned to be accessible for inspection and repair during operation, and structures with permanent ballast.[2)]	Spaces with solid ballast, void spaces, sea chests, small cofferdams.	3
1) Regular inspection by NDT. 2) No planned NDT inspection.		

7.3.1.13 The design criterion is:

$$D_D \leq 1.0.$$

7.3.1.14 The fatigue performance of welds can be improved by grinding. Grinding of welds will increase the calculated fatigue life of the welded connection if performed in accordance with the conditions specified in DNVGL-ST-0126 and in DNVGL-RP-C203.

Guidance note:

If welds are ground in order to increase the fatigue life, the nominal stress level will increase for the same calculated fatigue life. Possible fatigue cracks will therefore grow faster after the crack has initiated, if the weld has been ground.

The consequences of corrosion in the weld lines will also be larger in case of grinding. The S-N curve should be downgraded by one class as defined in DNVGL-RP-C203 and an S-N curve for free corrosion should be applied.

These two cases will both necessitate shorter time between inspections in order to keep the same safety level as for structures for which grinding is not performed.

To mitigate the larger consequences of corrosion in weld lines in case of grinding, corrosion protection measures such as coating and cathodic protection are recommended.

The designer is advised to improve the details locally by other means than grinding, or to reduce the stress range through design and keep the possibility of fatigue life improvement as a reserve to allow for possible increase in fatigue loading during the design and fabrication process.

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7.4 Accidental limit states

7.4.1 General

Floating support structures for wind turbines and their station keeping systems shall be designed against accidental limit states (ALS). Provisions and requirements for design against accidental limit states are given in [7.4.1.2] to [7.5.1.7].

7.4.1.1 The material factor γ_m for the ALS is 1.0, unless otherwise specified.

7.4.1.2 Design against the ALS shall be carried out for the following two situations:

— check of resistance of the structure against design loads
— check of post-damage resistance of the structure against environmental loads, for example when the structural resistance has become reduced by structural damage caused by loads such as those brought upon by the design collision, or when the station keeping system has become damaged by the loss of a mooring line or a tendon.

7.4.1.3 The overall objective of design against accidental loads is to achieve a structural system where the structural integrity is not impaired by the design accidental loads.

7.4.1.4 The design against accidental loads may be carried out directly by calculating the effects of the design accidental loads and checking that the resistance is not exceeded, or indirectly by designing the structure as tolerable to accidental events. One example of the latter consists of introducing compartmentalization of the floating structure in order to provide sufficient integrity to survive certain collision scenarios without further calculations.

7.4.1.5 The inherent uncertainty in the frequency and magnitude of some accidental loads, as well as the approximate nature of the methods for determination of the associated accidental load effects, shall be recognized and makes it essential to apply sound engineering judgment and pragmatic evaluations in design against accidental loads.

7.4.1.6 Typical accidental loads are listed in [4.7].

7.4.2 Post-accidental integrity after unintended change in ballast distribution

It is recommended to check the structural integrity of floaters consisting of multiple floating elements after the event of an unintended change in ballast distribution between the elements, e.g. as the result of an error in water filling of the ballast tanks. The check of the structural integrity can be based on an assumption of a particular prescribed trim or list of 10° or 20°. As an alternative to prescribing a particular trim or list, a trim or list resulting from the stability evaluation of the floater can be used as a basis. The assumed trim and list shall then be combined with wind and waves as appropriate.

7.4.3 Post-accidental integrity of a shared anchor

The load pattern of an anchor shared by two or more mooring lines or two or more tendons may change significantly, should one of the attached lines or tendons break or fail or otherwise be lost. The effect of such a changed load pattern shall be considered when the structural integrity of the anchor is assessed in the ALS in the post-accidental damaged condition.

7.5 Provisions for concrete structures

7.5.1 General

7.5.1.1 Floating concrete support structures for floating wind turbines shall be designed in accordance with DNVGL-ST-0126 together with either DNVGL-ST-C502 or EN 1992-1-1 as the basic design standard. The following subsections provides supplemental provisions to be considered during design. Unless it is specifically stated in the subsection that the text applies to one of the basic design standards, it applies to both basic design standards.

7.5.1.2
For requirements related to selection of concrete materials see [6.3].

7.5.1.3
For concrete design according to DNVGL-ST-0126 used together with EN 1992-1-1, Annex LL and Annex MM in EN 1992-2 can be applied for shell elements.

7.5.1.4
For requirements related to fabrication and construction of concrete structures see DNVGL-ST-C502.

7.5.1.5
For requirements related to in-service inspection and maintenance of concrete structures see DNVGL-ST-0126 and DNVGL-ST-C502.

7.5.1.6
Requirements for partial safety factors shall be taken in accordance with specifications given in DNVGL-ST-0126.

7.5.1.7
All elements subjected to an external/internal hydrostatic pressure difference shall as a minimum tightness requirement be designed with a permanent compression zone not less than the larger of:

— 0.25·h
— values as given in Table 7-6

Table 7-6 Depth of compression zone versus pressure difference

Pressure difference [kPa]	*Depth of compression [mm]*
< 150	100
> 150	200

See relevant load combinations for this check in [7.5.2.3] and [7.5.3.1].

7.5.1.8
Elements in an intact structure that shall remain watertight to ensure floatability of the structure, where loss of tightness may lead to loss of stability, shall be designed with a minimum membrane compressive stress equal to 0.5 MPa over the entire cross section. See relevant load combinations for this check in [7.5.3.1].

7.5.1.9
In elements that shall remain watertight to ensure floatability of the structure, the minimum reinforcement should be at least twice the value specified in the applied basic design standard and the spacing or the reinforcement bars should not be more than 300 mm.

7.5.1.10
Special care shall be taken to ensure watertightness in areas where fasteners or embedments are cast into the structure.

7.5.2 Accidental limit state

7.5.2.1
For elements that shall remain water tight to ensure floatability of the structure, where loss of tightness may lead to loss of stability, the additional requirements in [7.5.2.2] shall be met for the design condition "ALS for intact structure" combination (d), see Table 5-1, except that a load coefficient of 1.0 is used instead of 1.15 for the environmental loading.

7.5.2.2

The strain in the reinforcement shall not exceed the strain corresponding to a stress equal to $0.9f_{yk}$. Similarly, concrete compressive stresses shall be limited to $0.6f_{cck}$.

7.5.2.3

It shall be demonstrated that the structure can resist actions and has adequate stability after an ALS event until repairs or other precautions are undertaken to restore the structure serviceability. In this evaluation, the combination "ALS for damaged structure" combination (e), see Table 5-1, shall be considered. Watertightness as specified in [7.5.1.7] shall be documented for intact elements which shall remain tight to ensure stability of the damaged structure. For watertightness checks a load coefficient of 1.0 for the environmental loading shall be used instead of 1.15.

7.5.3 Serviceability limit state

7.5.3.1

Watertightness checks (compression zone and minimum membrane compressive stress, where applicable) shall be performed for the design condition using SLS combination, see [5.1.3]

7.5.3.2

The environmental loading (E) used to check for watertightness should be determined as the environmental load which it is not exceeded more than 100 times during the design life of the structure. It shall be demonstrated that in the periods where this load is exceeded, any potential leakage does not lead to the loss of stability of the structure.

7.6 Special provisions for specific floater types

7.6.1 Special provisions for semi-submersibles

7.6.1.1 General considerations with respect to methods of analysis and capacity checks of semi-submersibles are given in DNVGL-OS-C103 and DNVGL-RP-C103.

7.6.1.2 The maximum responses in a semi-submersible are often not governed by the maximum wave height and associated wave period. Waves with shorter period often give the highest response, e.g. periods that give maximum split forces between the columns, see Figure 7-2.

7.6.1.3 For preliminary design and for design in the survival conditions with parked turbine (DLC 6.x), design wave analyses may be performed in which the following responses in the hull should be considered:

— split forces (transverse, longitudinal or oblique sea for odd-columned semi-submersible)
— torsional moment about a transverse and longitudinal, horizontal axis (in diagonal or near-diagonal seas)
— longitudinally opposed forces between parallel pontoons (in diagonal or near-diagonal seas)
— longitudinal, transverse and vertical accelerations of deck masses.

It is recommended that a full spectral wave load analysis be used as basis for the final design in the survival condition. Hence, it is necessary to base the analysis on a scatter diagram of significant wave and peak period that also represents the ULS conditions for all relevant wave periods.

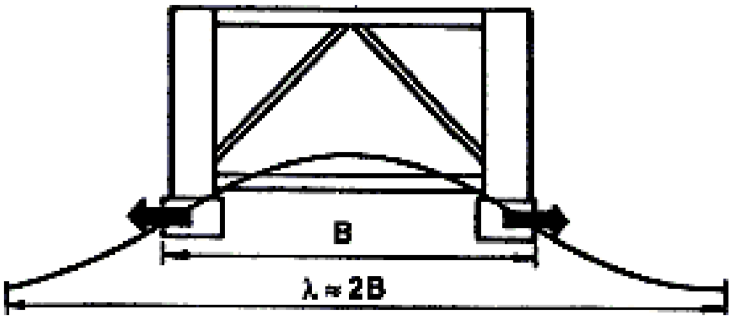

Figure 7-2 Split force between pontoons (from DNVGL-RP-C103)

7.6.1.4 In the ULS condition, positive air gap for main horizontal loadbearing members, buoyancy members and platforms should in general be ensured with a certain clearance for waves with a 50-year return period. When such positive air gap is not ensured, the effects of these waves on the structural components in question shall be documented and these components shall be designed against these effects. When sufficient air gap is not ensured for buoyancy members, wave overtopping might take place. Without such air gap, these members shall be designed against slamming loads. DNVGL-RP-C205 may be consulted for guidance regarding air gap. Local wave impact may be accepted in the ALS condition if such loads, including slamming loads, are adequately accounted for in the design.

7.6.1.5 Analysis undertaken to check air gap should be calibrated against relevant model test results when available, see DNVGL-OS-C103.

7.6.1.6 Column run-up load effects shall be accounted for in the design of horizontal structural elements and platforms above the still water line. The “run-up” loads should not be assumed to occur simultaneously with other environmental loads.

7.6.1.7 Requirements for the position mooring system are given in Sec.8. The following items relate directly to the mooring lines and the mooring equipment (windlass/winch, chain stopper, fairlead) supported on the hull and deck structure of the unit:

— Structural design procedure for the mooring lines, including mooring system analysis and design criteria formulated in terms of the ULS, the ALS and the FLS, are specified in Sec.8.
— Structural design procedure for mooring equipment such as windlass/winch, chain stopper, and fairlead are specified in DNVGL-OS-E301. The design of these components is based on a load equal to the characteristic breaking strength of the mooring lines.

Fairleads, winches, etc., and their local supporting structures, together forming part of the fixed position mooring system, shall withstand forces equivalent to 1.25 times the characteristic breaking strength of any individual mooring line, see DNVGL-OS-C103. The strength evaluation should be undertaken, utilizing the most unfavourable operational direction of the anchor line. In the evaluation of the most unfavourable direction, account shall be taken of relative angular motion of the unit in addition to possible line lead directions. The characteristic breaking strength is defined in [8.2].

7.6.1.8 The supporting structure influenced by the mooring forces, such as the support of winches and fairleads and the column shell between winch and fairlead, shall be designed for the following two main loading conditions:

a) Breaking load of one single mooring line:

$$F_{d,w1} = F_B \cdot \gamma_f$$

$F_{d,w1}$ = design load on windlass (corresponding to one mooring line)
F_B = characteristic breaking strength of one mooring line
γ_f = 1.25 (load factor, see [7.6.1.7])

The material factor γ_m is 1.0 in this case.

Operational loads from all mooring lines:

b) The design of all structural elements influenced by the mooring loads shall take into account relevant loads (ULS and ALS) found from the mooring analysis. The static and dynamic contributions to the mooring line forces should be considered for relevant application of load and material factors.

7.6.2 Special provisions for TLPs

7.6.2.1 General considerations regarding methods of analysis and capacity checks for TLPs and tendons are given in DNVGL-OS-C101 and DNVGL-OS-C105. Special considerations for tendons constructed from fibre ropes are given in DNVGL-OS-E303.

7.6.2.2 The maximum responses in a TLP are often not governed by the maximum wave height and associated wave period, see [7.6.1].

7.6.2.3 TLPs and their tendons shall be designed against both maximum and minimum water levels, defined respectively as high and low water levels with a specified return period, typically 50 years unless otherwise stated.

7.6.2.4 Special attention shall be placed on the design of tendons and their connection to the hull and their anchoring points. As a minimum, the following issues shall be considered in design of tendons:

— pretension (static tension)
— tide (tidal effects)
— storm surge (positive and negative values)
— tendon weight (submerged weight)
— overturning (due to current, thrust in operating condition, mean wind in survival condition, or drift load)
— set-down (due to current, thrust in operating condition, mean wind in survival condition, or drift load)
— WF tension (wave frequency component)
— LF tension (wind gust and slowly varying drift)
— ringing (HF response)
— springing
— seismic risk
— hull VIM influence on tendon responses
— tendon VIV induced loads.

7.6.2.5 Additional issues to be considered for tendons are:

— margins for fabrication, installation and tension reading tolerances
— allowance for foundation mispositioning
— foundation uplift and possible settlements
— loads due to spooling during transportation and storage of flexible tendons
— operational requirements (e.g. operational flexibility of ballasting operations).

7.6.2.6 Composite tendons shall be designed in accordance with DNVGL-ST-C501 with additional provisions as given in this standard.

7.6.2.7 Tendon failure may have substantial consequences and therefore the tendons shall be designed with sufficient safety margin. The floating structure, which is kept in position and supported by the tendons, shall be checked for the loss of one tendon in the ALS condition. Alternatively, instead of checking for the loss of one tendon in the ALS condition, the tendons shall be designed to a higher consequence class as specified in [2.2.1.5].

7.6.2.8 Tendon removal may be necessary for the purpose of maintenance, inspection or replacement. The tendon removal condition should be planned for. Consideration should be given to the expected frequency of tendon removal and to the length of period for which one tendon is likely to be out of service.

7.6.2.9 For the ALS condition, minimum tension in at least three corner groups of tendons shall remain non-negative in the required accidental environment as defined by [4.3.2.3]. If non-negative tension is not maintained in all corner groups in the required accidental environment, then a comprehensive coupled analysis of the tendon system performance under loss of tension shall be performed to demonstrate proper reengagement of the bottom connector with the foundation receptacle and adequate robustness against subsequent snatch loading. The analysis shall examine detailed load sequences induced in all components (top and bottom) on all tendons to ensure load capacities are not exceeded and components function as intended in order to prevent tendon disconnect.

7.6.2.10 Tendons and tendon components constructed from steel shall be designed against the FLS. Requirements for the design fatigue factor *DFF* for steel tendons are given in Table 7-7 as a function of consequence class.

Table 7-7 Minimum requirements for design fatigue factors, DFF, for steel tendons

Consequence class	*DFF*
1	3
2	10

7.6.2.11 For tendons constructed by fibre ropes, design against the FLS shall be carried out in accordance with requirements given in DNVGL-OS-E301, DNVGL-OS-E303 and DNVGL-RP-E305.

7.6.2.12 Guidelines for certification of tendons are given in DNVGL-OS-C105 App.A.

7.6.3 Special provisions for spars

7.6.3.1 General considerations with respect to methods of analysis and capacity checks of structural elements are given in DNVGL-OS-C106 and DNVGL-RP-C205.

7.6.3.2 In case that resonant or near resonant heave motion may occur, the theoretical predictions of heave motion should be validated against model test results. For spars, heave is in general not as important as pitch and roll.

7.6.3.3 The coupling between the yaw and pitch modes of motion shall be considered.

Guidance note:

A tilted rotor, in the case of a pitched floater, will result in yaw loads. This may be important when the yaw resistance is low, which can be the case for a spar.

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

7.6.3.4 If VIM suppression devices such as spiral strakes are attached to the hull, the increases that they cause in the drag and inertia loads shall be taken into account. This applies to the operational phase as well as to non-operational phases.

7.6.3.5 For spars, buoyancy cannot be calculated correctly by using simple Archimedes principles for the floater when waves are present. This can be realized by considering an infinitely long vertical cylinder in waves. In this case the passing wave will have no influence on the buoyancy because the dynamic pressure at the bottom end of the cylinder is zero and the buoyancy is therefore constant. For such systems, the total buoyancy effect can only be calculated correctly by pressure integration over all wet surfaces of the floating body. In some cases for a spar type floater the total variable vertical force due to the waves will be directed in the opposite direction of a simple Archimedes model. Correct modelling is therefore very important for deep spar systems.

7.6.3.6 Requirements for the position mooring system are given in Sec.8 and in [7.6.1].

SECTION 8 STATION KEEPING

8.1 Introduction

8.1.1 General

8.1.1.1 This section provides requirements for station keeping systems for floating wind turbine structures.

8.1.1.2 The station keeping system refers to the catenary or taut mooring systems of either chain, wire or fibre ropes for compliant support structures such as DDFs, or to the tendon systems of tethers for restrained support structures such as TLPs. In principle, station keeping could also be provided by use of dynamic positioning. The station keeping system is vital for keeping the wind turbine in position such that it can generate electricity and such that the transfer of electricity to a receiver can be maintained.

8.1.1.3 Unless otherwise specified, all structural components in the station keeping system of the floating support structure, such as mooring lines and tendons, shall be designed to consequence class 1 as is specified in [2.2.1.3]. This requirement refers to station keeping systems which have redundancy.

8.1.1.4 For station keeping systems without redundancy, all structural components in the station keeping system shall be designed to consequence class 2 as specified in [2.2.1.4]. This requirement reflects the risk for collision with adjacent wind turbine structures, should the floater happen to disengage from its station keeping system and float about within the wind farm that it constitutes a part of, for example in the event of a mooring line failure.

Guidance note:

In some cases it is obvious whether a station keeping system is redundant or not. For example, it is obvious that if failure of a tether in a TLP causes capsizing then the station keeping system of the TLP is a system without redundancy. In other cases, it is not so obvious whether a station keeping system is redundant or not. For example, failure of a slack mooring line in a three-line system, causing a large drift-off, does not necessarily imply a system without redundancy. In such cases, it may be necessary to carry out a qualification of the redundancy of the station keeping system, for example by documenting that the system is capable of withstanding loads in the damaged condition after an accident. For this purpose, characteristic environmental loads defined as 1-year loads can be assumed in conjunction with load factors for the ALS in the damaged condition in the relevant consequence class.

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

8.1.1.5 The requirement in [8.1.1.4] may be relaxed when there is no risk for collision with adjacent wind turbine structures. In this case, the structural components in the station keeping system shall be designed to the consequence class specified in [2.2.1.4].

Guidance note:

It is possible to lay out the wind farm and carry out the design of the station keeping system in such a manner that the risk for collisions with adjacent structures becomes reduced or maybe even eliminated. This may be done by keeping larger distances between adjacent structures, or by building redundancy into the station keeping system, including sufficiently large residual capacity of the damaged system, or by a combination.

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

8.1.2 Compliant floaters – general

8.1.2.1 The station keeping systems of compliant floaters are based on taut or catenary mooring lines that transfer the loads acting on the floater to anchors that are installed in the seabed soils. The anchor solution has to be decided on a case-by-case basis depending on the ground conditions on the actual site.

8.1.2.2 Optimization of mooring systems may lead to non-redundant systems where a mooring line failure may lead to loss of position and possible conflict with adjacent wind turbines. Redundancy considerations are therefore an important part of mooring design and form part of the basis for selection of the appropriate consequence class in accordance with [8.1.1.3] to [8.1.1.5].

Guidance note:

Safety factor requirements are dependent on consequence class and consequence class is dependent on consequence of failure which in turn is dependent on redundancy of system. In full analogy, DNVGL-OS-E301 has introduced the option of applying increased safety factors (SF) for mooring systems without sufficient redundancy relative to those required when there is redundancy in the system.

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

8.1.2.3 The fairlead is a device that guides a mooring line near the point where the mooring line is connected to the floater. The location of the fairlead has to be decided during the design process. In design, both LF and WF wind and wave induced responses have to be taken into account. For some concepts sufficient restoring stiffness in yaw may require use of 'crowfoots' or similar arrangements towards the floater and its fairleads.

8.1.2.4 Wear of mooring lines may form a problem for station keeping systems in the operational phase. It is recommended that wear of mooring lines is addressed in the design phase and accounted for in the design of the mooring lines.

8.1.2.5 Requirements for design of mooring lines are given in [8.2]. For further details of principles for design of mooring lines, see DNVGL-OS-E301.

8.1.3 Restrained floaters – general

8.1.3.1 The station keeping systems of restrained floaters are based on tendons that restrain one or more modes of motion. Systems with three or more tendons located with sufficient separation will be restrained not only in heave, but also in pitch and roll. Systems with only one central tendon will be restrained in heave and compliant in roll and pitch.

8.1.3.2 Floaters with restrained modes will typically experience responses in three ranges of frequencies, i.e. HF, WF and LF, and they are therefore expected to be more complex to analyse and design than structures with responses within just one limited frequency range.

8.1.3.3 Tendon systems for restraining the heave mode, and possibly also the pitch and roll modes, can be either metallic or composite. Steel and titanium are metals typically used for tethers in such systems.

8.1.3.4 For all tendon systems, the end terminations will usually be critical components which need special attention in design. This applies regardless of whether the tendon system is metallic or composite. Further complexity may be introduced if universal joints are introduced either at keel level or at seabed.

8.1.3.5 Requirements for design of tethers are given in [8.3] which refers to Sec.7. For further details of principles for design of metallic tethers, see DNVGL-OS-C105. For further details of principles for design of fibre ropes, see DNVGL-OS-E303.

8.1.3.6 When fibre rope is used for tendons and subjected to permanent tensile loads, time-dependent elongation of the fibre rope will take place owing to creep. This will imply an associated relaxation of the tension in the rope. Creep and its associated relaxation form a separate load case and shall be addressed in the design of tethers constructed from fibre rope. For this purpose the creep and its associated relaxation shall be combined with high or low water level, whichever is more unfavourable.

8.1.3.7 Redundancy considerations are an important part of tendon design and form part of the basis for selection of the appropriate consequence class in accordance with [8.1.1.3], [8.1.1.4] and [8.1.1.5].

Guidance note:

In analogy with the requirements in [8.1.1.3] to [8.1.1.5], for tendon systems without redundancy with respect to loss of a tether, the standard DNVGL-OS-C105 for TLP design recommends that load factors used for tether design be increased by 20% relative to those used for redundant systems.

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

8.2 Mooring lines

8.2.1 Introduction

8.2.1.1 This subsection provides requirements for design of mooring lines for floating wind turbine structures relying on either catenary mooring or taut mooring. Design of tendons for tension leg platforms is not covered in this subsection. For design of tendons, see [8.3].

8.2.2 Ultimate loads

8.2.2.1 The design tension T_d in a mooring line is the sum of two factored characteristic tension components $T_{c,mean}$ and $T_{c,dyn}$,

$$T_d = \gamma_{mean} \cdot T_{c,mean} + \gamma_{dyn} \cdot T_{c,dyn}$$

in which $T_{c,mean}$ = characteristic mean tension, $T_{c,dyn}$ = characteristic dynamic tension, and γ_{mean} and γ_{dyn} are load factors.

8.2.2.2 The characteristic mean tension $T_{c,mean}$ is defined as the mean part of the 50-year value of the line tension and is caused by pretension and mean environmental loads from static wind, current and wave drift. The characteristic dynamic tension $T_{c,dyn}$ is defined as the dynamic part of the 50-year value of the line tension and is caused by oscillatory low-frequency and wave-frequency effects.

Normally, the governing ULS cases for mooring design are found in environmental conditions with 50-year return period where the turbine is in parked condition, but conditions with maximum turbine thrust shall also be checked.

8.2.2.3 For estimation of $T_{c,mean}$ and $T_{c,dyn}$, a number of trial sea states defined in terms of H_s, T_p and U_{10} along a 50-year contour in (H_s, T_p, U_{10}) space shall be investigated. $T_{c,mean}$ and $T_{c,dyn}$ can then be selected from the particular sea state that yields the largest line tension response along the 50-year contour. An inverse FORM technique can be used to establish this 50-year contour. The joint probability distribution of H_s, T_p and U_{10} at the site of the mooring system is necessary for this purpose. The line tension in a mooring line is sensitive to the direction of the environmental loading. Therefore, when the investigation of the sea states along the environmental 50-year contour is carried out under an assumption of direction-independent environmental loading acting in the direction which is most unfavourable for the mooring line, this implies a conservatism is introduced such that the largest line tension response along the 50-year contour can be considered directly as an estimate of the 50-year value of the line tension without any need for multiplication with an inflation factor. $T_{c,mean}$ and $T_{c,dyn}$ can then be estimated by the mean part and the dynamic part, respectively, of this largest line tension response. Should it be desirable to account for directional distributions of wind and waves in a more detailed manner, rather than assuming direction-independent environmental loading acting in the most unfavourable direction, this can be done in accordance with a methodology described in NORSOK N-006.

8.2.2.4 The prerequisite for the procedure in [8.2.2.3] is that the 50-year value of the line tension is assumed to occur during a sea state along the 50-year environmental contour. However, the 50-year value of the line tension may not necessarily occur during a sea state along this contour, because sustained winds at the rated wind speed, where the operational thrust is often the highest, may cause the largest drift and the highest loads in the mooring system. For estimation of $T_{c,mean}$ and $T_{c,dyn}$, it is therefore important also to investigate the loads in the mooring system caused by sustained wind at the rated wind speed.

8.2.2.5 Details regarding how the characteristic mean tension and the characteristic dynamic tension can be established, e.g. from time series of tensile response in mooring lines, are given in DNVGL-OS-E301.

8.2.2.6 Requirements for load factors in the ULS and the ALS are given in Table 8-1 as a function of consequence class.

Table 8-1 Load factor requirements for design of mooring lines

Limit state	*Load factor*	*Consequence class*	
		1	*2*
ULS	γ_{mean}	1.3	1.5
ULS	γ_{dyn}	1.75	2.2
ALS	γ_{mean}	1.00	1.00
ALS	γ_{dyn}	1.10	1.25

8.2.3 Resistance

8.2.3.1 The characteristic capacity of a mooring line is defined in [8.2.3.5]. The premises for the definition are given in [8.2.3.2] to [8.2.3.4].

8.2.3.2 The characteristic capacity specified in [8.2.3.5] is defined on the following basis:

— The mooring line components shall be manufactured with a high standard of quality control, in accordance with recognized standards, such as DNVGL-OS-E302, DNVGL-OS-E303 and DNVGL-OS-E304.
— Careful control of all aspects of handling, transport, storage, installation and retrieval of the mooring lines is imperative to ensure that the capacity of the mooring lines is not compromised.

8.2.3.3 A mooring line is usually assembled from a large number of identical components together with a few connecting links such as line terminations. A chain line contains a large number of chain links. A long steel wire rope or a synthetic fibre rope may also be conceptually treated as a large number of wire rope segments. The strength of a long line is expected to be less than the average strength of the components that make up the line. This effect is taken into account in the definition of the characteristic capacity, see [8.2.3.5].

8.2.3.4 The following statistics are required for the strength of the components that make up the main body of the mooring line:

— μ_S, the mean value of the breaking strength of the component
— COV_S, the coefficient of variation of the breaking strength of the component.

8.2.3.5 The characteristic capacity of the body of the mooring line constructed from the component with the properties specified in [8.2.3.4] is defined by:

$$S_C = \mu_S \cdot (1 - COV_S \cdot (3 - 6 \cdot COV_S)); \ \mathrm{COV_S} < 0.10$$

This formulation is applicable for mooring lines consisting of chain, steel wire rope and synthetic fibre rope. The characteristic capacity S_C is sometimes referred to as the characteristic breaking strength of the mooring line.

8.2.3.6 When statistics of the breaking strength of a component are not available, then the characteristic capacity of the body of the mooring line may be obtained from the minimum breaking strength S_{mbs} of new components as:

$$S_C = 0.95 \cdot S_{mbs}$$

8.2.3.7 The statistical basis for the characteristic strength can also be applied to used components if breaking strength statistics are obtained for the used components by carrying out break load tests. However, the alternative basis using the minimum breaking strength should not be applied to used components without changing the reduction factor. To avoid a reduction in minimum breaking strength of 5%, the breaking tests of the mooring line segments can be performed to a load 5% higher than the specified minimum breaking strength. The number of tests shall be as required in DNVGL-OS-E302, DNVGL-OS-E303 and DNVGL-OS-E304.

8.2.3.8 When the strength distribution is based on test statistics, there will be statistical uncertainty in the results. The statistical uncertainty depends on the number of tests performed. In order to account for the statistical uncertainty, such that the target reliability is maintained, a cautious estimate of the characteristic capacity of the body of the mooring line shall be applied in design. The cautious estimate of the statistically uncertain characteristic capacity shall be taken as:

$$S_C^{\,*} = S_C \cdot (1 - 2 \cdot \frac{COV_S}{n})$$

in which COV_S is the coefficient of variation of the breaking strength of the component and n is the number of tests, not less than 5.

8.2.3.9 Other components in the mooring line such as connecting links and terminations shall be designed to have characteristic capacities which exceed the characteristic strength of the main body of the mooring line with a very high level of confidence.

8.2.3.10 For definition of the characteristic capacity of anchors for transfer of mooring line forces to the seabed soils, see Sec.9.

8.2.4 Design criterion

8.2.4.1 The design criterion in the ULS is:

$$S_C > T_d$$

in which the characteristic capacity S_C is to be replaced by $S_C^{\,*}$ when the strength distribution for the mooring line components is based on test statistics, see [8.2.3.8].

8.2.4.2 The design criterion in the ALS is:

$$S_C > T_d$$

in which the characteristic capacity S_C is to be replaced by S_C^* when the strength distribution for the mooring line components is based on test statistics, see [8.2.3.8], and in which T_d is established under an assumption of damaged mooring system in terms of one broken mooring line.

8.2.5 Fatigue design

8.2.5.1 Mooring lines shall be designed against fatigue failure. The design cumulative fatigue damage is:

$$D_D = DFF \cdot D_C$$

in which D_C is the characteristic cumulative fatigue damage caused by the stress history in the mooring line over the design life, and in which *DFF* denotes the design fatigue factor. Requirements for *DFF* for mooring chain are given in Table 8-2 as a function of consequence class. The design life of a mooring line will often be shorter than the design life of the floating structure and the wind turbine, hence implying that possible replacement of the mooring line during the design life of the floating unit is an implicit design assumption.

Table 8-2 Minimum requirements for design fatigue factors, DFF, for mooring chain

Consequence class	*DFF*
1	5
2	10

8.2.5.2 The characteristic cumulative fatigue damage D_C can be calculated by Miner's sum as outlined in Sec.7. For this purpose, characteristic S-N curves for chain and steel wire ropes, given in DNVGL-OS-E301 where they are referred to as design S-N curves, can be used. Characteristic S-N curves based on fatigue test data can also be used. For given stress S, the N-value of the characteristic S-N curve is defined as the 2.3% quantile in the distribution of the number of cycles to failure at this stress. When the characteristic S-N curve is based on limited test data it shall be estimated with at least 75% confidence.

8.2.5.3 The design criterion is:

$$D_D \leq 1.0.$$

8.2.5.4

If relevant, out-of-plane bending of in-chain links shall be accounted for according to DNVGL-OS-E301 and normal industry practice.

8.3 Tendons

8.3.1 Steel tendons

Requirements for design of steel tendons against the ULS and ALS are given in Sec.7. Load factor requirements for steel tendons are given in [5.1.1]. Requirements regarding tendon slack are given in [8.3.3].

8.3.2 Fibre rope tendons

8.3.2.1 The design tension, T_d, in a fibre rope tendon shall be calculated in accordance with specifications of characteristic loads given in Sec.4 and requirements for load factors and load combinations given in [5.1.1].

8.3.2.2 The characteristic tensile capacity, R_c, of a tendon constructed from fibre ropes shall be taken as the 2.5% quantile in the probability distribution of the tensile capacity in an arbitrary cross section along the tendon. The design tensile capacity of the tendon is:

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

where the material factor γ_m shall be taken equal to at least γ_m = 1.5 under an assumption of ductile material behaviour and not more than 10% coefficient of variation of the tensile capacity.

8.3.2.3 The design criterion in the ULS is:

$$R_d > T_d$$

8.3.2.4 Fibre rope tendons shall be designed against fatigue failure. Requirements for fatigue design of fibre rope tendons shall be established on a case-by-case basis. Useful guidance can be found in DNVGL-OS-E301 and DNVGL-OS-E303.

8.3.2.5 In designing fibre ropes with respect to cyclic endurance, it is important to notice the difference from fatigue of steel structures. Fibre rope durability is usually driven by the static tension level, see [8.3.2.6]. The structure shall be designed so as to inhibit internal abrasion, and the non-detrimental effect of cyclic loading with respect to static holding capacity (3-T, i.e. tension, time, temperature) shall be demonstrated.

8.3.2.6 Fibre rope tendons shall be designed against failure for sustained loads caused by pretension and sustained environmental loads. The accumulated static loads shall be used for establishing sufficient design margin against creep failure or stress rupture, whichever is applicable. Useful guidance can be found in DNVGL-OS-E301 and in DNVGL-OS-E303. Further recommendations are provided in DNVGL-RP-E305.

Guidance note:

The ability to withstand creep failure or stress rupture, depending on type of failure, is referred to as 3-T capacity, see DNVGL-OS-E303.

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

8.3.3 Tendon slack

8.3.3.1 Tendons shall be designed against slack in the ULS. As an alternative to designing against slack in the ULS, tendons may be designed against slack in the ALS, where the characteristic environmental dynamic compressive force in the tendon is defined as the dynamic compressive force with return period 500 years.

Guidance note:

A tendon is considered slack when $\gamma_{f,G} \cdot T_{pretension,c} + \gamma_{f,E} \cdot T_{dynamic,c} \leq 0$, in which $\gamma_{f,G}$ is the load factor for permanent loads, $\gamma_{f,E}$ is the load factor for environmental loads, $T_{pretension,c}$ is the characteristic pretension in the tendon (positive), and $T_{dynamic,c}$ is the characteristic environmental dynamic compressive force in the tendon (negative). Requirements for the load factors are given in [5.1.1]. Usually the load factor set denoted ULS (b) is the set that governs the design against tendon slack.

It may be too conservative to design against slack by requiring $\gamma_{f,G} \cdot T_{pretension,c} + \gamma_{f,E} \cdot T_{dynamic,c} > 0$. It is advisable to carry out an initial analysis in which the tendon is considered as a spring in a straight line between the connection positions. This is a conservative approach with respect to compression in the tendon, because the tendon will in reality deflect and thereby maintain positive tension much longer. However, the spring approach can be nonconservative with respect to the integrity of the hull, because restoring is maintained also after the tendon has become slack. Adequate design against slack can be demonstrated by a detailed nonlinear dynamic analysis in the time domain.

Note that the event of tendon slack does not necessarily take place during an extreme storm event, but may occur during operation, for example in an emergency stop case.

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

8.3.3.2 The requirement in [8.3.3.1] can be waived when the following two conditions are fulfilled:

— at least three corner groups of tendons are in tension in order to maintain floating stability at any time
— less than half of the corner groups of tendons are slack.

When these two conditions are fulfilled and the tendons are not designed against slack, a comprehensive redundancy analysis shall be performed to evaluate the effect of the loss of tension in one corner group on the tendon system and its supporting structures. The analysis shall demonstrate that structural integrity is maintained when tendons with negative tension are removed. As an alternative to this analysis, model tests may be performed to demonstrate that the structural integrity is maintained.

8.3.3.3 When temporary tendon tension loss is permitted in the ULS in accordance with [8.3.3.2], tendon dynamic analyses shall be conducted to evaluate the effect of the tension loss on the complete tendon system and its supporting structures. Alternatively, model tests may be performed. The reasoning behind this requirement is that loss of tension could result in detrimental effects to components such as tendon body, connectors, flex elements, floating structure and anchor foundations. Such detrimental effects include fatigue of various structural components and cyclic strength degradation of anchor foundations.

SECTION 9 DESIGN OF ANCHOR FOUNDATIONS

9.1 Introduction

9.1.1 General

9.1.1.1 This section deals with the geotechnical design of the anchoring systems that transfer loads between (1) the mooring lines or the tendons of the station keeping system and (2) the seabed soils. The section also deals with the design of grouted rock anchors for transfer of loads from the station keeping system to a seabed consisting of rock rather than soil.

9.1.1.2 The requirements for geotechnical foundation design given in DNVGL-ST-0126 apply to the geotechnical design of the anchoring systems of floating wind turbine structures with the exceptions, deviations and additional requirements specified in this section. The requirements for material factors specified in DNVGL-ST-0126 apply unless otherwise specified in this section.

9.1.1.3 This section deals with the design of the following types of anchor foundations of relevance for anchoring of floating wind turbine units:

— pile anchors
— gravity anchors
— suction anchors
— free-fall anchors
— fluke anchors
— plate anchors
— prestressed rock anchors.

Owing to their modes of operation, fluke anchors and plate anchors are expected only to be suitable as individual anchor points for anchoring of single wind turbine structure, but not as shared anchor points for anchoring of multiple wind turbine structures.

9.1.1.4 Type-specific requirements for anchor design are given in [9.2] through [9.8].

9.1.1.5 The analysis of anchor resistance shall be carried out for the ULS and the ALS and it shall be carried out in accordance with the safety requirements in Sec.2. Due consideration shall be given to the specific aspects of the particular anchor type in question and the current state of knowledge and development. For grouted rock anchors, analysis of the anchor resistance may also be necessary for the SLS.

9.1.1.6 The anchor is defined as a load-bearing structure and shall be designed against geotechnical anchor failure in the ULS and in the ALS.

9.1.1.7 Design against the ULS is intended to ensure that the anchor with its geotechnical anchor resistance can withstand the loads arising in an intact station keeping system under extreme environmental conditions. Likewise, design against the ALS is intended to ensure that the anchor can withstand the loads arising in an intact station keeping system under accidental load conditions. Design against the ALS is also intended to ensure that the damaged station keeping system retains adequate capacity if one mooring line, one tendon or one anchor should accidentally fail for reasons outside the designer's control.

9.1.1.8 It should be kept in mind that the design lifetime for anchors may be considerably longer than for the floaters and turbines they are to support, such that they should be prepared and designed for several hook-ups over time.

9.1.2 Anchor load

9.1.2.1 The design force, T_d, acting on the anchor and arising from line tension in a mooring line, which is hooked up to the anchor, shall be taken as equal to the design line tension in the mooring at the interface between the mooring line and the anchor, as resulting from calculations in accordance with specifications given in [8.2.2]. When more than one mooring line is hooked up to the anchor, the design force T_d, acting on the anchor, shall be calculated with due consideration of design force contributions from all mooring lines.

9.1.2.2 The design force, T_d, acting on the anchor and arising from tension in a tendon, which is hooked up to the anchor, shall be taken as equal to the design tension in the tendon at the interface between the tendon and the anchor, as resulting from calculations in accordance with specifications of characteristic loads given in Sec.4 and requirements for load factors and load combinations given in [5.1.1]. When more than one tendon is hooked up to the anchor, the design force T_d, acting on the anchor, shall be calculated with due consideration of design force contributions from all tendons.

9.1.2.3 Some anchors are used as individual anchor points for anchoring of single wind turbine units only, whereas other anchors serve as shared anchor points for anchoring of multiple wind turbine units. Caution must in general be exercised when assessing the load pattern at an anchor and determining the design force T_d, in particular when the anchor is a shared anchor point for which the load pattern can be expected to be complex.

9.1.2.4 The load pattern of a shared anchor may change significantly, should one of the attached lines or tendons break or fail or otherwise be lost. The effect of such a changed load pattern shall be considered when the structural and geotechnical integrity of the anchor is assessed in the ALS in the post-accidental damaged condition.

9.1.3 Anchor resistance

9.1.3.1 Unless otherwise stated, the characteristic anchor resistance is defined as the mean anchor resistance as set up by the supporting soils or rock.

9.1.3.2 The characteristic value of a soil property or an anchor resistance shall be estimated from site-specific soil data. Further, the characteristic value shall be established as a function of both the design state in question and the available soil data. Relevant statistical methods may be used for estimation of characteristic values of soil properties. If such methods are applied, the characteristic value shall be estimated with confidence. The confidence level applied shall reflect the quality and quantity of the soil data and the complexity of the soil conditions.

Guidance note:

For additional guidance on the determination of characteristic soil properties, please see DNVGL-RP-C212. Typical levels of confidence used in geotechnical design would be 75%, 90% or 95%. For additional guidance on the correct use of statistical methods for determining characteristic soil properties, please refer to DNVGL-RP-C207.

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

9.1.3.4 Unless otherwise stated, the design anchor resistance is:

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

in which R_c is the characteristic geotechnical anchor resistance and γ_m is a material factor.

9.1.3.5 Requirements for the material factor are given for the individual anchor types in [9.2] through [9.8].

9.1.4 Design criterion

The design criterion is:

$$T_d \leq R_d$$

9.1.5 Effects of cyclic loading

9.1.5.1 The effects of cyclic loading on the soil properties shall be considered in the geotechnical design of anchors for the station keeping systems of floating wind turbines.

9.1.5.2 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.

9.1.5.3 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 by an emergency shutdown, for several successive storms, and for any other wave and wind load condition that may influence the soil properties.

Guidance note:

The load cycles and the load rate felt by the anchor will cause cyclic degradation, reducing the soil strength and stiffness, or load rate effects with a positive effect on the soil strength. For one-way cyclic loading in clay where a stress cycle does not involve stress reversal, it is common to see a favourable combined effect of the cyclic degradation and the rate effect. The combined effect is a cyclic shear strength which is somewhat larger than the static shear strength of the soil. For two-way cyclic loading in the clay where a stress cycle involves stress reversal, the combined effect of cyclic degradation and rate effect is usually less favourable, consisting of a cyclic shear strength which is somewhat smaller than the static shear strength of the soil.

For a pile or an anchor which is used as a shared anchor for more than one mooring line, e.g. with one mooring line from each of many wind turbine floaters, the loading may be rather complex. One-way cyclic loading which is commonly assumed for an anchor, which serves only one mooring line and which is subjected to a permanent mean load from this line, cannot necessarily be assumed. Some degree of two-way cyclic loading and its less favourable combined effect of cyclic degradation and rate effect can be expected.

Low-frequency load cycles are often encountered in mooring lines and their supporting anchors. A less beneficial rate effect may be expected from low-frequency stress cycles than from wave-frequency and high-frequency stress cycles in the supporting soils.

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9.1.6 Shared anchor points

9.1.6.1 Shared anchor points for anchoring of multiple wind turbine floaters are feasible for some anchor types. For design of shared anchor foundations, the load contributions from all mooring lines shall be accounted for.

9.1.6.2 In general, anchors will mainly be subject to one-way cyclic loading, whereas for a shared anchor point the cyclic loading can be one-way loading in one direction and two-way loading in another direction. This more complex cyclic loading pattern for shared anchors shall be considered when assessing cyclic soil strength degradation and the anchor holding capacity.

9.1.7 Scour and scour prevention

9.1.7.1 The risk for scour around an anchor foundation shall be taken into account unless it can be demonstrated that the surrounding soils will not be subject to scour for the expected range of water particle velocities. This is relevant for anchors with exposed parts above the seabed, such as pile anchors, suction anchors and, in some instances, fluke anchors.

9.1.7.2 The effect of scour, where relevant, shall be accounted for in accordance with at least one of the following methods:

— adequate means for scour protection are placed around the exposed part of the anchor foundation as early as possible after installation
— the anchor foundation is designed for a condition where all materials, which are not scour-resistant, are assumed removed
— the seabed around the anchor is kept under close surveillance and remedial works to prevent further scour are carried out shortly after detection of significant scour
— note that depending on the details of a project in question, monitoring might also be required for the first two approaches.

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

9.1.7.4 For details about scour and scour prevention in general, see DNVGL-RP-C212. For details about scour at a vertical pile from a fluid mechanics perspective, see DNVGL-ST-0126.

9.2 Pile anchors

9.2.1 General

9.2.1.1 Pile anchors shall be designed in accordance with the relevant requirements given in DNVGL-ST-0126. Useful guidance is given in DNVGL-RP-C212.

9.2.1.2 Long-term effects of creep under permanent tension shall be accounted for in design.

9.2.2 Material factors

9.2.2.1 The soil material factors to be applied to the characteristic resistance of pile anchors shall not be taken less than:

γ_m = 1.3 for the ULS
γ_m = 1.0 for the ALS

regardless of consequence class and regardless of the material factors specified in DNVGL-ST-0126.

9.2.2.2 When there is little or no possibility for redistribution of loads from one pile anchor to another, or from one group of pile anchors to another group of pile anchors, larger material factors than those given in

[9.2.2.1] shall be used in design. This may apply, for example, to pile anchors for TLPs and DDFs. In such cases the material factor shall not be taken less than γ_m = 1.7 for ULS design.

Guidance note:

Useful guidance can be found in the commentary annexes of API RP 2T.

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9.2.3 Anchors in chalk

9.2.3.1 Pile anchors in chalk form a special case of so-called rock-socketed shafts; i.e. laterally loaded drilled piles in chalk.

9.2.3.2 For design of pile anchors in chalk, attention should be paid to the fact that over time, the uppermost chalk may become crushed owing to cyclic loading of the pile and the associated motions. Such crushing will lead to a redistribution of the pile forces which will then be transferred to the chalk further down along the pile. This will imply a risk of a progressive development of failure along the pile. It might be necessary to design the pile anchor under an assumption of zero resistance from the chalk in a zone of a few metres measured vertically from the seabed down.

Guidance note:

The vertical extent of the zero-resistance zone below the seabed is expected to depend on the pile diameter, the pile stiffness and the quality of the chalk.

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9.2.3.3 When a p-y curve approach is used for the design of pile anchors in chalk, a p-y curve model should be used which has a residual resistance at large pile deflections, which is lower than the peak resistance at some moderate deflection. Caution must be exercised in providing adequate strength and stiffness data for such a p-y curve model.

9.3 Gravity anchors

9.3.1 General

9.3.1.1 Gravity anchors shall be designed in accordance with the relevant requirements given in DNVGL-ST-0126.

9.3.1.2 The capacity against uplift of a gravity anchor shall not be taken larger than the submerged mass of the anchor. However, for anchors furnished with skirts penetrated into the seabed soils, the capacity contribution from soil frictional resistance along the skirts may be included, but only as a capacity for carrying the dynamic part of the load, and only with due consideration of creep effects in the selection of the material factor on the characteristic frictional resistance. In certain cases such anchors may be able to resist cyclic uplift loads by the development of temporary suction within their skirt compartments. In relying on such suction one shall make sure that there are no possibilities for leakage, e.g. through pipes or leaking valves or channels developed in the soil, which could prevent the development of the suction.

9.3.2 Material factors

9.3.2.1 The material factors to be applied to the characteristic resistance of gravity anchors shall not be taken less than

γ_m = 1.3 for the ULS

γ_m = 1.0 for the ALS

regardless of consequence class.

9.4 Suction anchors

9.4.1 General

9.4.1.1 Suction anchors are vertical cylindrical anchors with open or closed top, which are installed by self-weight penetration followed by application of underpressure (suction) in the closed compartment. The failure mechanism in the clay around an anchor will depend on various factors such as the load inclination, the anchor depth-to-diameter ratio, the depth of the load attachment point, the soil strength profile, and whether the anchor has an open or a closed top. Suction anchors are widely used in clays where it is easy to establish suction within the anchor owing to the low permeability of the soil. This type of anchor is in some cases also used in other types of soils.

Guidance note:

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

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

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9.4.1.2 Anchor resistance is a function of soil properties such as the undrained shear strength and is composed of a horizontal and a vertical resistance.

9.4.1.3 Coupling between vertical and horizontal resistances occurs when the failure mechanism is a combination between vertical and horizontal translation modes. The coupling may reduce the vertical and horizontal resistance components at failure, and the resulting resistance will be smaller than the vector sum of the uncoupled maximum vertical and horizontal resistances. This is illustrated by an example in Figure 9-1. $R_{V,max}$ denotes the vertical resistance in pure vertical loading, $R_{H,max}$ denotes the horizontal resistance in pure horizontal loading, R_V is the vertical force, and R_H is the horizontal force. The blue curve designates the resistance under combined loading.

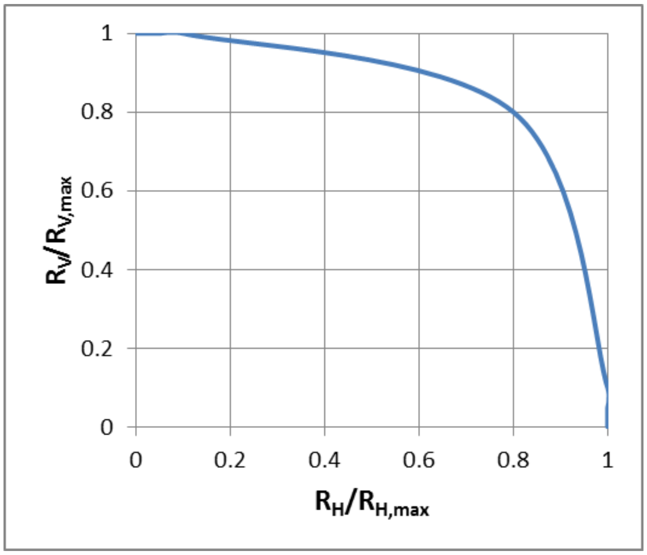

Figure 9-1 Schematic resistance diagram for suction anchor, example

9.4.1.4 Recommendations for geotechnical design and installation of suction anchors in clay are provided in the recommended practice DNVGL-RP-E303. The design method outlined in this recommended practice makes use of a relatively detailed resistance analysis. Many existing analytical methods will meet the analysis requirements specified in this recommended practice.

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

9.4.1.6 In the calculation of the anchor resistance, strength anisotropy and the effects of cyclic loading on the undrained shear strength shall be accounted for.

9.4.1.7 The characteristic anchor resistance shall be taken as the anchor resistance that results from a characteristic undrained shear strength defined as the mean value. When the characteristic undrained shear strength is estimated from limited data, the estimate shall be a cautious estimate, see [9.1.3.2].

9.4.1.8 Seabed impact landing and subsequent penetration by self-weight shall be addressed in terms of required water evacuation areas to avoid excessive channelling and/or global instability during installation. Possible retrieval by means of overpressure shall also be considered. Load factors for loads associated with impact landing, suction to target penetration depth and possible retrieval by means of overpressure shall be taken in accordance with Table 5-1. For loads associated with permanent removal after service life the load factors may be taken equal to 1.0.

9.4.1.9 If suction anchors are used in other types of soils than clay, due consideration shall be given to installation issues. It shall be demonstrated that it will be possible to establish suction during installation by designing the anchor in such a manner that it has a sufficient self-weight penetration depth, and it shall be verified that no wash-out of the soil within the skirt compartments will take place because of the suction. In addition, the effects of the soil permeability on the ability of the anchor to resist long-term suction under operation should be addressed.

9.4.2 Material factors

9.4.2.1 The soil material factors to be applied to the characteristic resistance of suction anchors shall not be taken less than

γ_m = 1.2 for the ULS regardless of consequence class
γ_m = 1.0 for the ALS for consequence class 1
γ_m = 1.2 for the ALS for consequence class 2.

9.5 Free-fall anchors

9.5.1 General

9.5.1.1 Free-fall anchors are projectile-shaped objects which are installed by dynamic penetration of the soil by means of a free-fall velocity achieved by the effects of gravity. Given the shear strength of the soil, the velocity of the free-fall anchor at impact governs the final penetration depth and the final penetration depth governs the pull-out capacity of the anchor.

Guidance note:

So-called torpedo piles form one type of free-fall anchors.

Free-fall anchors are typically dropped from a height of 50 to 100 metres above the seabed. The seabed impact velocity can reach about 25 m/s and the penetration depth is maximized by optimizing the pile geometry, the centre of gravity (COG) and the installation procedure, while minimizing the as-installed inclination.

The applicability of free-fall anchors depends on the soil conditions on the site.

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9.5.1.2 Site-specific soil conditions shall be established in accordance with requirements set forth in DNVGL-ST-0126. The anchor penetration at completion of installation shall be recorded together with the final anchor inclination in the soil and the pile azimuth. It shall then be checked that these are within the acceptable tolerances from the design.

Guidance note:

Double integration of accelerometer data may be used to determine the achieved penetration depth of the anchor into the seabed.

The achieved penetration depth can also be verified by inspecting pre-marked positions on the mooring line by means of an ROV.

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9.5.1.3 For prediction of the impact velocity of the free-fall anchor, all forces acting in the free-fall phase through water shall be considered, including self-weight, buoyancy and drag. For prediction of the final penetration depth, all forces acting in the penetration phase shall be considered, including buoyancy, soil frictional resistance, end bearing and drag.

9.5.1.4 For prediction of the final anchor resistance all contributing components shall be considered, including submerged weight, reverse end bearing, soil friction, resistance on components such as pad-eyes and flukes, and time-dependent set-up effects owing to thixotropy and consolidation. Effects of cyclic loading on the final anchor resistance shall also be taken into account.

9.5.1.5 For design of the anchor, the design load and the corresponding line angle at the pad-eye shall be predicted. This can be done in accordance with principles outlined in DNVGL-RP-E301.

9.5.1.6 Three-dimensional finite element analysis, assuming representative upper and lower bound stiffnesses of the surrounding soil, can be used to predict the structural response of the anchor when subjected to the design axial and lateral load components at the pad-eye. The lower bound stiffness may be governing for the lateral response of the anchor.

9.5.1.7 The characteristic anchor resistance is defined as the mean anchor resistance.

9.5.1.8 Currently, no formal design procedure exists for free-fall anchors, but the design principles for piles as outlined in DNVGL-RP-C212 can normally be adopted. For deeply penetrated anchors the vertical (axial) resistance component at the pad-eye governs the anchor capacity, whereas for more shallowly penetrated anchors the lateral resistance component (and lateral displacements) may govern this capacity. So-called "stick-ups" shall be avoided, i.e. full penetration is required with no part of the installed anchor exposed above the seabed.

9.5.1.9 A plan for an appropriate hook-up time shall be established with adequate allowance for set-up effects to take place before hook-up of the mooring lines or tendons is initiated. In case the hook-up takes place before full soil reconsolidation is achieved, the design shall account for the soil softening due to clay remoulding caused by the pile installation as well as for the degree of clay reconsolidation at the time of hook-up. This can be assessed analytically by means of cavity expansion theory and pore pressure dissipation calculations or by means of FEM analyses. Alternatively, instrumented tests on anchors installed in the target area prior to the design phase can be used to document the soil reconsolidation after installation.

9.5.1.10 Installation acceptance criteria and contingency procedures shall be established before the installation takes place. The installation acceptance criteria shall as a minimum consider the pile installation inclination and its azimuth with respect to the mooring centre, the anchor penetration depth and the degree of set-up predicted at the time of hook-up to the mooring system. The anchor penetration depth, inclination and azimuth shall be verified by measurements as required in [9.5.1.2].

9.5.2 Material factors

9.5.2.1 The soil material factors to be applied to the characteristic resistance of free-fall anchors shall not be taken less than

γ_m = 1.3 for the ULS

γ_m = 1.0 for the ALS

regardless of consequence class.

9.6 Fluke anchors

9.6.1 General

9.6.1.1 Design of fluke anchors shall be based on recognized principles in geotechnical engineering supplemented by data from tests performed under relevant site and loading conditions. The penetration resistance of the anchor line shall be taken into consideration where deep penetration is required to mobilize reaction forces.

9.6.1.2 Fluke anchors shall normally be used only for unidirectional horizontal load application. A fluke anchor is therefore unlikely to be suitable as a shared anchor point for more than one floating wind turbine unit. Some uplift may be allowed under certain conditions. A recommended design procedure for fluke anchors is given in DNVGL-RP-E301.

9.6.1.3 The characteristic anchor resistance in the touchdown point consists of a characteristic installation resistance $R_{i,c}$ in conjunction with characteristic post-installation resistance effects. Provided that the target installation tension T_i is reached and verified by reliable measurements during the installation of the anchor, the characteristic installation resistance $R_{i,c}$ can be set equal to the target installation resistance T_i. The characteristic post-installation effects are defined as the expected values of the respective post-installation effects. In case the anchor is designed not to drag after installation, the post-installation effects consist of the thixotropy effects, the consolidation effects, the cyclic loading effects and the line friction on the seabed. In case anchor movement by additional drag is allowable after installation, the post-installation effects consist of the additional resistance gained through additional anchor drag and penetration, the cyclic loading effects and the line friction on the seabed. It is recommended that the characteristic post-installation effects are estimated with caution, in particular in soils where shallow anchor penetration is expected at the end of installation.

9.6.1.4 If additional anchor movement by additional drag is allowed, the magnitude of this movement shall be assessed and it shall be verified that the associated additional drag is acceptable with respect to offset of the floating structure and with respect to the available safety factors against the breaking strength of the affected mooring lines.

9.6.1.5 When the anchor is designed not to drag after installation, the design anchor resistance R_d in the touchdown point is defined as

$$R_d = R_{i,c} + \frac{\Delta R_{set\text{-}up,c} + \Delta R_{cy,c} + \Delta R_{fric,c}}{\gamma_m}$$

When the anchor design relies on additional drag after installation, the design anchor resistance R_d in the touchdown point is defined as

$$R_d = R_{i,c} + \frac{\Delta R_{drag,c} + \Delta R_{cy,c} + \Delta R_{fric,c}}{\gamma_m}$$

$\Delta R_{set\text{-}up,c}$	=	denotes the characteristic post-installation resistance owing to thixotropy and consolidation,
$\Delta R_{cy,c}$	=	denotes the characteristic post-installation effect due to cyclic loading,
$\Delta R_{fric,c}$	=	denotes the characteristic line friction on the seabed,
$\Delta R_{drag,c}$	=	denotes the characteristic resistance gained by further drag and penetration of the anchor after installation, and
γ_m	=	is a material factor.

For details, see DNVGL-RP-E301.

9.6.2 Material factors

The soil material factors to be applied to the characteristic post-installation resistance of fluke anchors shall not be taken less than

γ_m = 1.3 for the ULS under normal loads, regardless of consequence class

γ_m = 1.0 for the ALS for consequence class 1

γ_m = 1.3 for the ALS for consequence class 2.

9.7 Plate anchors

9.7.1 General

9.7.1.1 Design methodologies for plate anchors such as drag-in plate anchors, push-in plate anchors, drive-in plate anchors and suction embedment plate anchors, shall be established with due consideration of the characteristics of the respective anchor types, including how the anchor installation affects the in-place conditions.

9.7.1.2 Plate anchors are normally to be used only for unidirectional load application. A plate anchor is therefore unlikely to be suitable as a shared anchor point for more than one floating wind turbine unit.

9.7.1.3 The anchor resistance is a function of the undrained shear strength at the penetration depth in question, of the plate area and of the effect of cyclic loading. This is outlined in detail in DNVGL-RP-E302.

9.7.1.4 The characteristic resistance of a plate anchor at a specific penetration depth is defined as the mean anchor resistance at this depth. For calculation of the mean anchor resistance, the mean undrained shear strength at this depth shall be used in conjunction with the expected effect of cyclic loading.

9.7.1.5 Recommended design procedures for plate anchors are given in DNVGL-RP-E302.

9.7.2 Material factors

The soil material factors to be applied to the characteristic resistance of plate anchors shall not be taken less than

γ_m = 1.4 for the ULS regardless of consequence class

γ_m = 1.0 for the ALS for consequence class 1

γ_m = 1.3 for the ALS for consequence class 2.

9.8 Prestressed rock anchors

9.8.1 General

9.8.1.1 Prestressed rock anchors generally consist of steel elements, or tendons, such as steel bars or wire-rope strands, grouted in a predrilled hole in the rock. The tendons shall be designed and the grouting performed in such a manner that appropriate unbonded and bonded lengths are ensured. Once the grout is fully cured, the rock anchor is tensioned, i.e. the anchor is inherently stressed prior to hook-up of the floater, hence the term prestressed rock anchor. See Figure 9-2 for an overview of the prestressed rock anchor components and features.

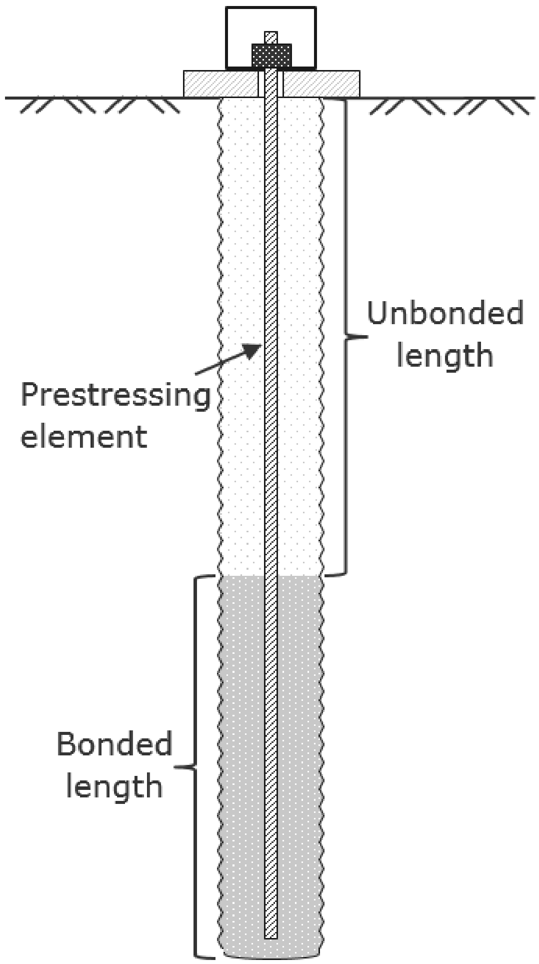

Figure 9-2 Prestressed rock anchor, components and features

Guidance note:

Prestressing is achieved by applying a tensile load at the top of the anchor. This tensile load is defined as a function of the characteristic operating load and is known as the lock-off load. In traditional on-shore rock anchors, this is achieved by either tightening a nut on a threaded anchor bar or by installing wedges which secure the individual strands of a multi-strand anchor. Although these concepts may not be relevant for use with a floating wind turbine, they do serve a function in explaining the concept of the system. In addition, the function of the bonded and unbonded lengths should not be overlooked. The overall purpose of these features is to ensure that the load is transferred to a depth which ensures mobilization of a competent rock mass, sufficient to resist the design loads. This is achieved by preserving an unbonded length, i.e. an initial length of rock anchor which is free to move relative to the surrounding grout/rock.

For guidance regarding the design of rock anchors, particularly related to the bonded and unbonded lengths, the proof-load and the lock-off load, see current standards of practice for prestressed rock anchors, such as PTI DC35.1.

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9.8.1.2 The properties of tendons, connections and grout are usually well defined, whereas those of the rock are usually not. Ground investigations are therefore necessary in the assessment of the suitability of prestressed rock anchors for a site in question.

9.8.1.3 As a minimum, ground investigations shall include sufficient rock coring and sampling to determine ground conditions such as rock formation and groundwater, and to identify any adverse layers or seams. Sufficient field and laboratory testing shall be used in order to classify the rock, for example rock quality designation (RQD), geologic strength index (GSI) and rock mass rating (RMR). In addition, geophysical core-logging may be useful in identifying more realistic in-situ properties, such as density, stiffness characteristics, permeability and cracks. Laboratory testing shall be used to quantify compressive and tensile strengths and stiffness properties.

9.8.1.4 The grout used for rock anchors is generally micro-fine or ultra-fine cements and/or sodium silicate which can be injected. For selection of grout, the following properties of the grout shall be considered:

— pumpability (workability)
— durability of high strength and low porosity for long-term life
— resistance to salinity of seawater.

9.8.1.5 For design of a prestressed rock anchor against failure in the ULS, failure in the following modes and in combinations of them shall be considered:

— failure of grout
— failure of contact between grout and rock
— failure of contact between grout and steel element
— failure of rock mass
— failure of steel element.

9.8.1.6 The anchor will experience not only axial load but also considerable lateral loads (owing to floater motions) which may govern the design.

9.8.1.7 Effects of cyclic loading shall be considered in design. Cyclic loading may lead to crushing of the grout and/or the rock surrounding the anchor and may thereby cause a degradation of the final anchor resistance.

9.8.1.8 The anchor shall be designed on the basis of the ground conditions. The design can be carried out on the basis of experience in similar conditions. The design can also be carried out on the basis of investigation tests, i.e. tests on instrumented anchors, which are installed in the target area prior to the design phase, and which are not used for any other purpose. Alternatively, the design can be confirmed based on the performance of suitability tests. These are similar to investigation tests, but are performed on the actual rock anchors to be used for mooring the floating wind turbine. Both tests shall demonstrate that creep is negligible and confirm the load carrying capacity of the prestressed rock anchors at both the proof and lock-off loads.

Guidance note:

The investigation tests are sacrificial by nature and should be tested to grout/rock failure. For these tests, it may be relevant and acceptable to use a larger tendon in order to ensure grout/rock failure as the governing failure mode.

The suitability tests are not sacrificial.

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9.8.1.9 The anchor shall be designed under due consideration of appropriate measures to ensure corrosion protection for the life of the tendon. Post grouting after proof test and lock-off may be used to further increase corrosion protection and load transfer.

9.8.1.10 Special attention shall be placed when installing anchors in sloping ground conditions. The depth of the anchor bond length shall be such that it does not negatively impact the slope.

9.8.1.11 Challenges involved with grouting and grouting procedures under water, particularly in deep waters, shall be considered.

Guidance note:

If the tendons are to be installed after drilling, special attention should be paid in order to maintain an open hole, particularly in mixed soil/rock conditions. This is particularly important in order to ensure sufficient bond is achieved between the grout and the surrounding rock.

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9.8.1.12 A post-installation test program shall be defined and carried out. In the case of anchors which will support TLPs, each installed prestressed rock anchor shall be included in the test program. The test program shall be carried out after curing of the grout has taken place and the anchor has reached its full capacity. The test program shall include a set of suitability tests, including a pretension to the design lock-off tension. The lock-off tension is the permanent tensile lock-off load that the anchor will be subject to before any mooring line or tendon is hooked up to the anchor. The set of suitability tests should also include a creep test at the design lock-off tension. A proof test in which the anchor is subjected to an additional test tension beyond the design lock-off tension shall also be carried out. The additional test tension is a tensile load used temporarily only during the proof testing.

Guidance note:

It is recommended that at least three suitability tests are performed. The purpose of the suitability tests is to more closely define the creep and load-loss characteristics at proof and lock-off loads.

EN 1997-1, EN 1537 and PTI DC35.1 may be consulted for guidance regarding how to proof test.

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9.8.1.13 Anchor acceptance criteria and contingency procedures shall be established before the post-installation test program is executed and may be based on experience and accepted past practice for prestressed rock anchors. The anchor acceptance criteria shall at a minimum consider the pile installation inclination and its azimuth with respect to the mooring centre and the tensile capacity as documented by the test program.

9.8.1.14 The presence of a lock-off tension distinguishes the tensioned rock anchor addressed in [9.8] i from the pile anchor addressed in [9.2] . Other than this, the major difference between [9.2] and t[9.8] is that [9.2] is based on limit state design, whereas [9.8] is based on design by testing.

SECTION 10 FLOATING STABILITY

10.1 Introduction

10.1.1 General requirements

10.1.1.1 Floating stability implies a stable equilibrium and reflects a total integrity against downflooding and capsizing. Satisfactory floating stability of floating wind turbine units is necessary in order to support the safety level required for the involved structures.

10.1.1.2 This section provides minimum requirements for static floating stability of floating wind turbine units.

Guidance note:

Static floating stability needs to be demonstrated in the early stages of design. This is merely a matter of determining where the centre of gravity (COG) of the floating unit should be located in order to ensure that the unit is stable. Satisfactory dynamic floating stability for dynamically amplified loads will then be demonstrated by a successful and properly executed dynamic analysis at a later stage during design, hence confirming that the determined location of the COG is adequate for the purpose of the floating unit.

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10.1.1.3 Sufficient floating stability is an absolute requirement in the intact condition. This applies to the operational phase as well as any temporary phases. Requirements for intact stability are given in [10.2]

10.1.1.4 For unmanned floating wind turbine units, i.e. for units which are unmanned during extreme environmental conditions and during normal operation of the wind turbine, sufficient floating stability is not a requirement in the damaged condition, but an option which should be considered. Requirements for damaged stability, when damaged stability is opted for, are given in [10.3].

10.1.1.5 Location and design of manholes and hatches for access to the tower shall be carefully evaluated and designed such that water ingress will not take place.

10.1.1.6 The requirements for stability of the floating unit shall be met in the following service modes as deemed applicable for the unit or concept in question:

— operation, i.e. a normal working condition with the wind turbine operating
— temporary conditions, i.e. transient conditions such as installation and changing of draught
— survival condition, i.e. conditions during extreme storms
— transit, in particular tow-out.

10.1.1.7 The stability requirements in [10.2] and [10.3] are based on righting moment curves with acceptance criteria expressed in terms of requirements for the area of the righting moment curve relative to the area of the wind heeling moment curve and, in special cases, in terms of a simple requirement for the location of the metacentric height GM. As an alternative to demonstrating sufficient stability by fulfilling the requirements for stability given in [10.2] and [10.3], the stability may be assessed by establishing the restoring forces against pitch and roll from

— water plane area
— buoyancy
— station keeping system (only for evaluation of intact stability, not for evaluation of damaged stability)

and may be accepted as sufficient provided adequate acceptance criteria in terms of energy requirements for the restoring forces can be established and met. Restoring forces can be calculated in accordance with specifications given in DNVGL-RP-C205 Sec.7 [2].

10.1.1.8 Maximum vertical centre of gravity (VCG) limit curves shall be prepared in accordance with the stability requirements specified in this section.

10.1.1.9 The loading of the floating unit at all intended service draughts and modes shall be within the limits of maximum allowable VCG curves.

10.1.1.10 In order to determine the VCG of the actual loading conditions, the lightweight and its centre of gravity must be known. These properties shall either be obtained by an inclination test of one prototype unit or by analytical calculation. The analytical calculation may consist of a detailed lightweight calculation supplemented by a lightweight survey confirming the calculated weight for the prototype. When units of the same design are manufactured at more than one yard, dependent of the method selected, a lightweight survey or inclination test shall be carried out at each yard.

10.1.1.11 For evaluation of sufficient floating stability it is crucial to assess the wind loads. The wind loads consist of wind loads on the rotor in combination with wind loads on the tower and other wind-exposed parts of the support structure.

Guidance note:

The wind loads can be calculated in simplified manner by means of a method given in DNVGL-OS-C301 Ch.2 Sec.1 [2]; however, this method will be insufficient for assessment of the thrust force on the rotating rotor. The rotor thrust can be calculated in simplified manner by means of momentum theory supplemented by an additional term due to drag. Blade element momentum theory can also be applied to predict the rotor thrust.

For preliminary assessments for horizontal axis turbines, the rotor thrust can be estimated by the following expression,

$$F_{thrust} = \frac{1}{2} \cdot \rho \cdot C_T \cdot A_{rotor} \cdot U_{10}^2$$

in which ρ is the density of air, U_{10} is the far-field 10-minute mean wind speed, C_T is the thrust coefficient and A_{rotor} is the swept area of the rotor. The swept area is $A_{rotor}=\pi R^2$, where R is the rotor radius. In the absence of data, typical values for the thrust coefficient C_T can be extracted from Figure 10-1; however, it is important to appreciate that the thrust coefficient will vary dependent on the wind turbine type.

Caution should be exercised when evaluating the floating stability under the influence of wind loads. Consistency between wind speeds is the issue. The thrust coefficient of the rotor is provided for the 10-minute mean wind speed at hub height, whereas the stability criteria are often defined for a one-minute wind speed at 10 m height. Conversion of wind speeds may be necessary. Conversion formulas are available in DNVGL-RP-C205.

Alternative calculation of thrust force for vertical axis turbines may be acceptable.

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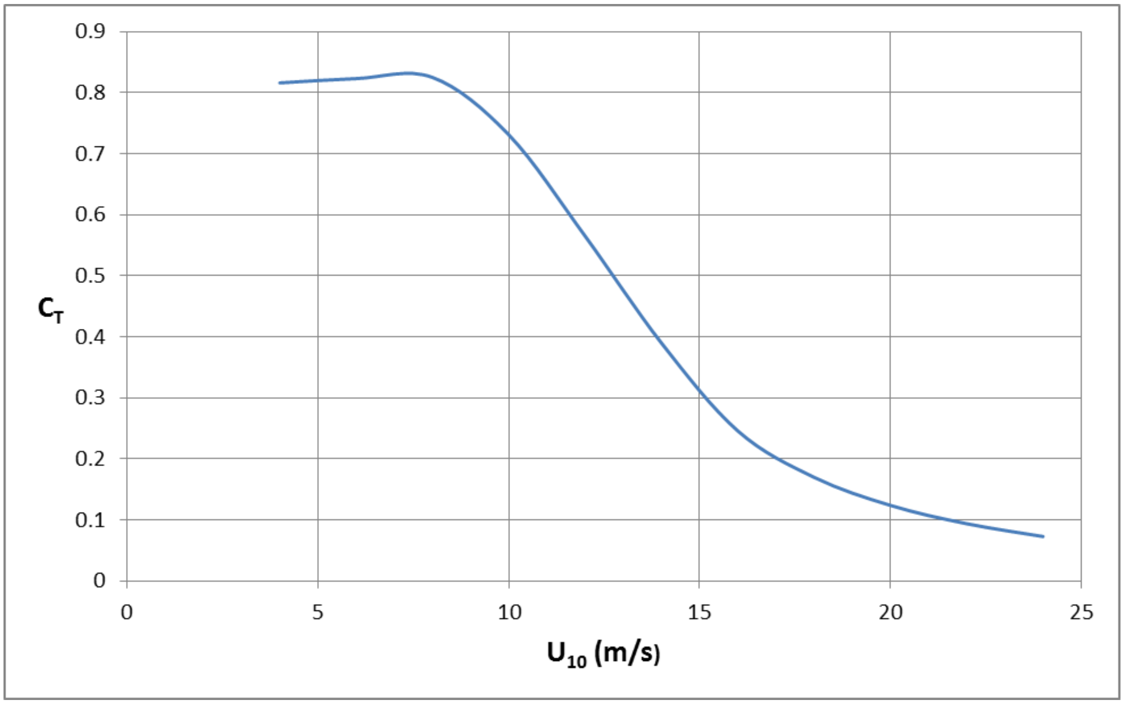

Figure 10-1 Thrust coefficient vs. 10-minute mean wind speed; rotor diameters 100-120 m

10.1.1.12
An emergency response plan shall be worked out. The emergency response plan may be included as part of the operational manual for the floating unit. The emergency response plan shall contain sufficient information about evacuation of personnel from the floating unit.

10.1.1.13 A stability manual shall be worked out. The stability manual may be included as part of the operational manual for the floating unit. The stability manual shall contain sufficient information to enable operation of the floating structure in compliance with the applicable stability requirements. Main dimensions, maximum draught, maximum trim and deadweight data with masses and positions of centres of gravity shall be stated in the stability manual with reference to a clearly defined reference system and baseline. The maximum VCG is the most important information to be stated in the stability manual. The stability model, consisting of a sketch of volumes contributing to buoyancy in intact and damaged stability calculations, shall be included in the manual. The following items shall be included in the stability manual:

— main dimensions and general particulars
— maximum VCG curve
— typical loading conditions
— instruction on stability calculation and ballast operation (if applicable)
— hydrostatic data
— list of openings
— lightweight data
— tank capacity (including void spaces)
— tank tables.

10.1.1.14 Material requirements for permanent ballast used for stability purposes are given in Sec.6.

10.2 Intact stability

10.2.1 General

10.2.1.1 The floating structure shall be capable of maintaining stability during operation of the wind turbine at the wind speed that produces the largest rotor thrust. The floating structure shall also be capable of maintaining stability during standstill of the wind turbine in severe storm conditions. These conditions shall be defined in consistence with the metocean conditions of the target environmental class. The procedures recommended and the approximate length of time required, considering both operating conditions and transit conditions, shall be contained in the stability manual.

Guidance note:

During operation of the wind turbine, the largest rotor thrust force usually occurs at the rated wind speed. However, depending on the wind turbine type and on the relative magnitudes of rated wind speed, 50-year wind speed and turbulence, the largest rotor thrust force may occur at other wind speeds than the rated wind speed. Conditions near the cut-out wind speed may be relevant, for example because of the combined effects of thrust from the operating turbine and large waves.

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10.2.1.2 The wind heeling moments applied in the stability calculation for a wind speed equal to the wind speed that produces the largest rotor thrust may be worked out in accordance with the methods specified in [10.1.1.13], assuming that the rotor plane is perpendicular to the direction of the wind.

10.2.1.3 The wind heeling moments applied in the stability calculation during standstill of the wind turbine in severe storm conditions may be worked out in accordance with the methods specified in [10.1.1.13], assuming that the rotor plane is perpendicular to the direction of the wind.. A wind speed of 51.5 m/s (100 knots) shall be assumed for the intact stability calculation. Should metocean data from the relevant site reveal that this wind speed will never occur at the hub height, a lower wind speed may be applied, based on the available data. To obtain sufficient stability also during fault of the yaw system during severe storm conditions, it will be necessary to also consider different turbine yaw values (between 0° and 360°) when calculating the wind heeling moments; however, a wind speed of 36 m/s (70 knots) may be assumed for this situation.

Guidance note:

During storm conditions, it is common to pitch the blades to feather while maintaining the rotor plane perpendicular to the wind direction. However, some turbines will instead yaw the rotor 90° out of the wind so that the rotor plane is parallel to the wind direction. To analyse wind heeling moments for rotor planes not perpendicular to the wind, more advanced models than specified in [10.1.1.11] may be needed.

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10.2.1.4 In calculating the wind heeling moments, the lever of the wind overturning force shall be taken vertically from the centre of pressure of all wind-exposed surfaces to the centre of lateral resistance of the underwater body of the unit or to the level of the mooring line attachment points, whichever is the lower. For a unit in the temporary condition floating free of mooring restraint, the lever shall be taken vertically from the centre of pressure of all wind-exposed surfaces to the centre of lateral resistance of the underwater body of the unit.

10.2.1.5

Relevant wind speeds are to be superimposed from any direction. The righting moment curves shall represent the most critical heeling axis, i.e. the heeling axis where the maximum VCG-value for the stability requirements reaches its lowest value.

10.2.2 Barges

10.2.2.1 Requirements for intact stability of barges are given in [10.2.2.2].

10.2.2.2 The area under the righting moment curve to the second intercept or downflooding angle, whichever is less, shall be equal to or greater than 140% of the area under the wind heeling moment curve to the same limiting angle. See Figure 10-2.

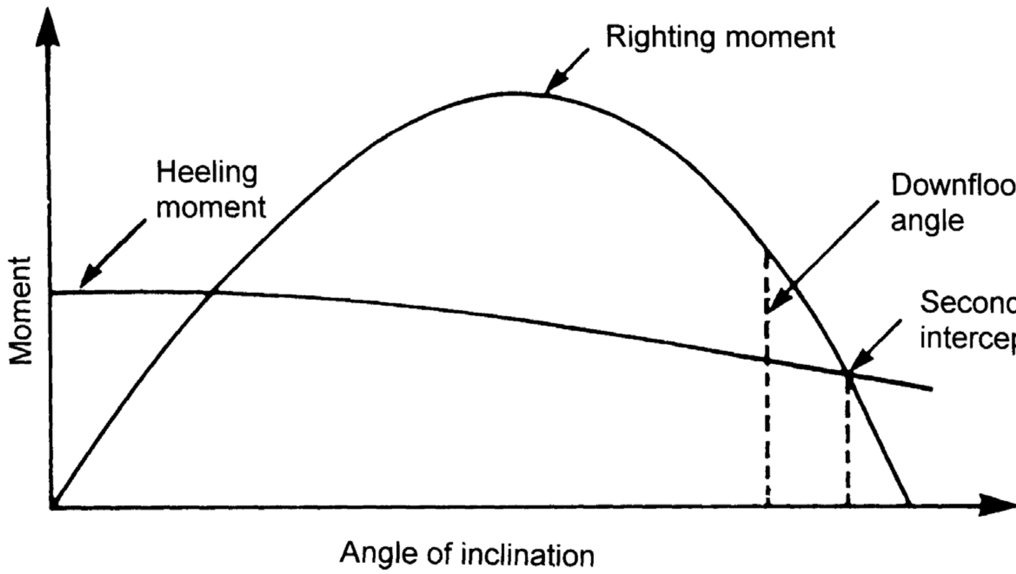

Figure 10-2 Righting moment and wind heeling moment curves

10.2.3 Semi-submersibles

10.2.3.1 Requirements for intact stability of semi-submersibles are given in [10.2.3.2] and [10.2.3.3].

10.2.3.2 The area under the righting moment curve to the second intercept or downflooding angle, whichever is less, shall be equal to or greater than 130% of the area under the wind heeling moment curve to the same limiting angle.

10.2.3.3 The righting moment curve shall be positive over the entire range of angles from upright to the second intercept.

10.2.4 Spars

10.2.4.1 Requirements for intact stability of spars are given in [10.2.4.2].

10.2.4.2 The metacentric height GM shall be equal to or greater than 1.0 m. The metacentric height GM is defined as the difference between the vertical level of the metacentre and the vertical level of the centre of gravity and shall be calculated on the basis of the maximum vertical centre of gravity VCG.

10.2.5 Tension leg platforms

10.2.5.1 Requirements for intact stability of TLPs are given in [10.2.5.2].

10.2.5.2 The intact stability of a TLP in temporary free-floating conditions during construction, tow-out and installation shall, in general, satisfy requirements applicable to semi-submersibles as defined in [10.2.3].

10.2.5.3 Some TLPs in the permanent in-place condition are only vertically restrained with ability to pitch and roll in operation, whereas others are restrained in heave, roll and pitch. The stability in the permanent in-place condition of a TLP is typically provided by the pretension and stiffness of the tendon system, rather than by the water-plane area.

10.2.5.4 The stability analysis shall demonstrate that the system is sufficiently constrained by the tendon system and that it is safe from overturning in all foreseeable environmental conditions. It is therefore important to monitor the weight change and Centre of Gravity (COG) shift in various operational modes and environmental conditions.

10.2.5.5 The allowable horizontal shift of the COG shall at a minimum be calculated for the following three load conditions or operational modes:

— still water
— operating environment
— survival environment.

10.2.5.6 The allowable shift of COG may be presented as an envelope relative to the originally calculated COG.

10.2.5.7 The allowable weight and horizontal COG shift shall be calculated based on maximum and minimum allowable tendon tension. Variation of the vertical COG, which results in changes in motion response and dynamic loads, shall be taken into account in the calculation.

10.2.5.8 An inclination test or an analytical calculation as specified in [10.1.1.11] shall be conducted to accurately determine the weight and the COG of the TLP. This applies to the TLP in the compliant, freely floating condition prior to restraining the unit by assembling the tendons. It suffices that the inclination test or the analytical calculation is carried out on one prototype unit only.

Guidance note:

The weight and the COG of a TLP are important with respect to the floating stability of the TLP, in particular during tow-out. In the permanent installed condition, the stability is governed by the tendons, not by the COG. However, TLPs are in general weight sensitive and the response in the tendons is sensitive to the location of the VCG. It is therefore important to have good control of the weight and the location of the VCG for these reasons also. This becomes particularly important for TLP concepts with ringing responses.

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10.3 Damage stability

10.3.1 General

10.3.1.1

For unmanned units, damaged stability is not a requirement. However, it is advisable to perform nonlinear collision analysis or to meet the damage stability requirements described in this subsection.

10.3.1.2
A nonlinear collision analysis showing that the floating structure remains watertight after an unintended collision by the maximum authorized service vessel in the ALS condition can be used as support when not designing for damage stability. For this purpose, the service vessel shall be assumed to be free 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 authorized service vessel may be considered.

Guidance note:

The maximum authorized service vessel is the largest expected vessel used in daily operation. Data for the maximum authorized service vessel, including impact velocities of a laterally drifting vessel, are usually given in the design basis for structural design of the floating unit. Note that supply vessels may grow in size over the years and the impact load may become substantial.

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10.3.1.3 Additional compartmentalization as commonly used for manned structures, most often to provide for damage stability, is usually not necessary for unmanned units, unless more stringent requirements in this respect are specified by government authorities or operators. Regardless of whether additional compartmentalization is adopted or not, the design can be combined with remote monitoring of the water level inside internal compartments as a mean to control that floating stability remains intact.

Guidance note:

The key issue when assessing the need for additional compartmentalization is that flooding can be a problem and a sufficiently low probability of filling is necessary if damage stability shall be deemed unnecessary.

The choice between multiple compartments and only one compartment in the floater hull structure can be based on a cost-benefit analysis. In the cost-benefit analysis, the probability of flooding must be evaluated and so must the monetary equivalent of the consequences of flooding, usually consisting of total loss of one floating wind turbine unit and temporary damage to part of the cable system. In the evaluation of total loss, also the environmental impact and salvage cost should be included. This must be balanced against the cost of constructing the unit with damage stability, e.g. in terms of compartmentalization of the hull. Based on this it can be assessed whether the probability of flooding is so large that damage stability is profitable and should be opted for.

Flooding is expected to take place either following a boat collision which leads to penetration of the hull, or it takes place through doors and other openings. Doors and other openings can be located high enough that they will not serve as leakage points. Only small service vessels are assumed to approach the structure and form the potential cause for damage. The associated probability of leakage and probability of subsequent filling are expected to be so low that – according to the cost-benefit analysis – only one compartment will suffice for most unmanned units.

However, for large wind turbines in excess of approximately 10 MW, the cost associated with the loss of one unit because of lack of stability may be so large that implementation of measures, e.g. in terms of additional compartmentalization, to avoid total loss may be considered beneficial.

Collision rings in the splash zone form one example of additional compartmentalization that can be implemented to provide an extra barrier against the consequences of damage. The need for a collision ring in the splash zone may be evaluated with basis in local legislation and requirements with due consideration of

— manned or unmanned structure
— type of substructure material (concrete, steel, composite, or a combination)
— size of maximum authorized service vessel
— resistance against service vessel impacts.

Cofferdams behind fairleads form another example of additional compartmentalization that can be used to provide an extra barrier against the consequences of damage.

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10.3.1.4 For assessment of stability in the damaged condition, it shall be demonstrated that the floating structure complies with the requirements of [10.3.2] to [10.3.5] by calculations, which take into consideration the proportions and design characteristics of the structure and the arrangements and configuration of the damaged compartments. In making these calculations it shall be assumed that the unit or installation is floating free of mooring restraints. It can also be assumed that the rotor can be in idling mode and that the blades can be pitched to feather. Requirements for assumptions regarding the extent of damage are given in [10.3.6].

10.3.1.5
Relevant wind speeds are to be superimposed from any direction. The righting moment curves shall represent the most critical heeling axis, i.e. the heeling axis where the maximum VCG-value for the stability requirements reaches its lowest value.

10.3.2 Barges

10.3.2.1 Requirements for damaged stability of barge-shaped structures, when applicable, are given in [10.3.2.2] and [10.3.2.3].

10.3.2.2 The floating structure shall have sufficient freeboard, buoyancy and stability to withstand in general the flooding of any one compartment in any operating or transit condition consistent with the damage assumptions set out in [10.3.6].

10.3.2.3 The floating structure should have sufficient reserve stability in a damaged condition to withstand the wind heeling moment based on a wind speed of 25.8 m/s (50 knots). In this condition the final waterline, after flooding, should be below the lower edge of any downflooding opening.

10.3.3 Semi-submersibles

10.3.3.1 Requirements for damaged stability of semi-submersibles, when applicable, are given in [10.3.3.2] and [10.3.3.3].

10.3.3.2 The floating unit shall provide sufficient buoyancy and stability in any operating or transit condition to withstand the flooding of any watertight compartment wholly or partially below the waterline in question, which is a pump room, a room containing machinery with a salt water cooling system or a compartment adjacent to the sea. The assessment of the buoyancy and stability shall include the wind heeling moment and shall take the following considerations into account:

1) In order to avoid progressive collapse, no progressive filling or inclination shall take place. Any potential downflooding point shall therefore be kept out of the water. This in particular implies that any opening below the final waterline shall be made watertight.
2) A range of positive stability shall be provided, beyond the calculated angle of inclination in these conditions, of at least 7°.

10.3.3.3 When the buoyancy and the stability are assessed , it is recommended that the wind heeling moment is based on a wind speed of 25.8 m/s (50 knots).

10.3.4 Spars

10.3.4.1 Requirements for damaged stability of spars, when applicable, are given in [10.3.4.2] and [10.3.4.3].

10.3.4.2 The floating structure shall have sufficient freeboard and be subdivided by means of watertight decks and bulkheads to provide sufficient buoyancy and stability to withstand a wind heeling moment induced by a wind speed of 25.8 m/s (50 knots) superimposed from any direction in any operating or transit condition, taking the following considerations into account:

1) In order to avoid progressive collapse, no progressive filling or inclination shall take place. Any potential downflooding point shall therefore be kept out of the water. This in particular implies that any opening through which progressive flooding may occur below the final waterline shall be made watertight. In addition, openings within 4 m above the final waterline shall be made weathertight, i.e. water shall not penetrate into the unit through these openings in any sea conditions.

2) The righting moment curve shall have, from the first intercept to the lesser of the extent of weathertight integrity required by (1) and the second intercept, a range of at least 7°.
3) The angle of inclination after the damage set out in [10.3.6] shall not be greater than 17°

10.3.4.3 Within this range, the righting moment curve shall reach a value of at least twice the wind heeling moment curve, where both the righting moment and the wind heeling moment shall be measured at the same angle, see Figure 10-3.

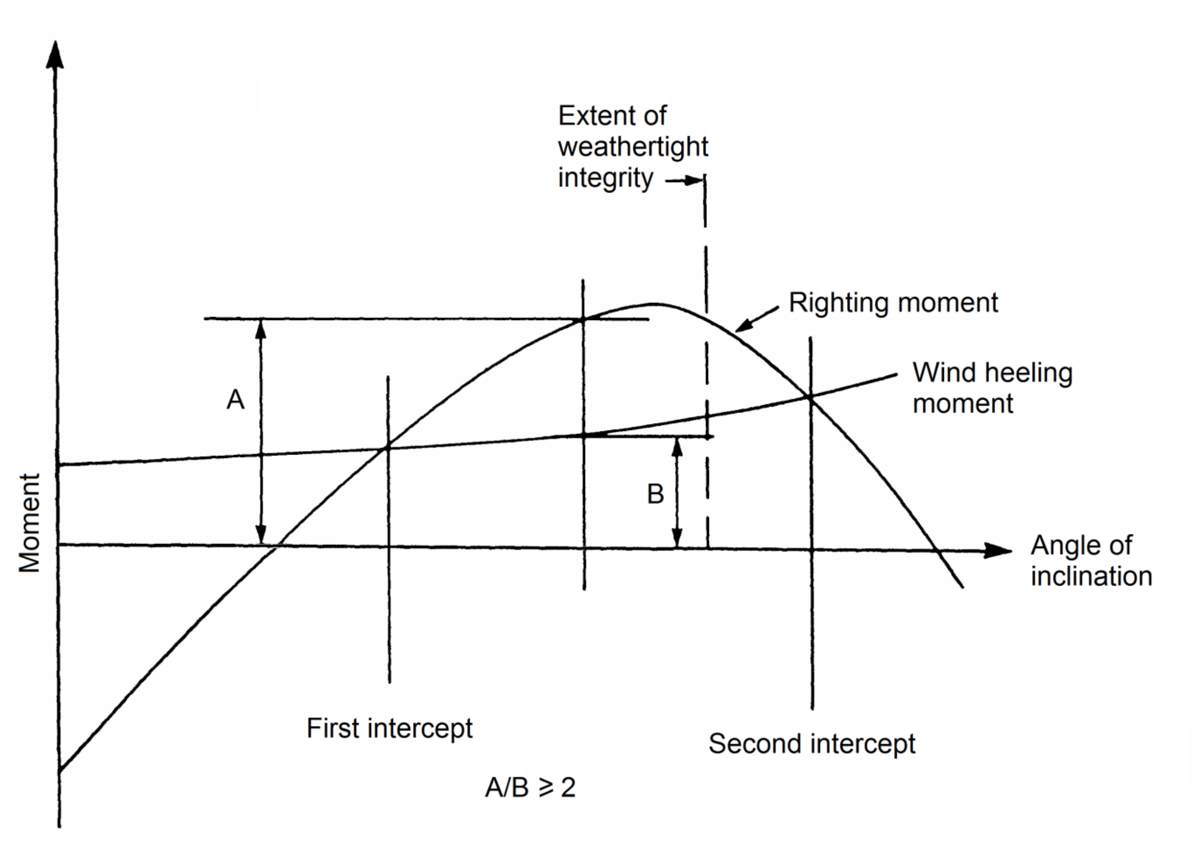

Figure 10-3 Righting moment and wind heeling moment curves

10.3.5 Tension leg platforms

10.3.5.1 Requirements for damaged stability of TLPs, when applicable, are given in [10.3.5.2] through [10.3.5.6].

10.3.5.2 The damaged stability of a TLP in temporary free-floating conditions during construction, tow-out and installation shall, in general, satisfy requirements applicable to semi-submersibles as defined in [3.3].

10.3.5.3 In-place stability of a TLP under an accidental event shall be measured by minimum and maximum tension criteria using the same principle as defined in [10.2.5]. The allowable weight and the COG shift envelope shall be established for the damaged condition using the same procedure as defined in [10.2.5]. The characteristic values of environmental loads and the load and resistance factor requirements are given in Sec.4 and Sec.5. The time lag between the occurrence of the damage and the restoration of the stability should be considered in the design to assure the safety of the TLP structure during this period.

10.3.5.4 In assessing the structural strength and adequacy of the tendon tension, the following flooding scenarios shall be assumed:

1) Any one tendon compartment
2) All compartments that could be flooded as a result of damages that are as minimum 1.5 m deep and 3.0 m high occurring at any level between 5.0 m above and 3.0 m below the still water line.

   Due consideration shall be given to the size of the service vessels and other potential collision scenarios before deciding the extent of the damage.
3) No vertical bulkhead shall be assumed damaged, except where bulkheads are spaced closer than a distance of one eighth of the column perimeter at the still water line, measured at the periphery, in which case one or more of the bulkheads shall be disregarded.

10.3.5.5 All piping, ventilation systems, trunks, etc., within the extent of damage shall be assumed damaged. Positive means of closure shall be provided at watertight boundaries to preclude the progressive flooding of other spaces that are intended to be intact.

10.3.5.6 Unintended flooding of hull and deck shall be treated as an accidental event. Tendon flooding shall also be treated as an accidental event.

10.3.6 Extent of damage

10.3.6.1 Assumptions for extent of damage are necessary in order to assess damage stability.

10.3.6.2 In assessing the damage stability of barges as specified in [10.3.2], the following extent of damage shall be assumed to occur between effective watertight bulkheads:

— horizontal penetration: 1.5 m
— vertical extent: from the baseline upwards without limit.

The distance between effective watertight bulkheads or their nearest stepped portions which are positioned within the assumed extent of horizontal penetration shall be not less than 3.0 m; however, wherever there is a smaller distance, one or more of the adjacent bulkheads shall be disregarded. Wherever damage of a smaller extent than the one specified above results in a more severe condition, such smaller extent shall be assumed.

10.3.6.3 In assessing the damage stability of semi-submersibles, the unit shall sustain the damage described in [10.3.3.2], i.e. damage to a single compartment adjacent to sea (or pump and machinery room). The vertical damage extent may be taken equal to the vertical damage extent specified for column-stabilized mobile offshore units, see DNVGL-OS-C301.

10.3.6.4
In assessing the damage stability of spars, the unit shall comply with the damage stability survival requirements in assuming flooding of any single watertight compartment located between 3 m below and 5 m above the waterline corresponding to the maximum draught.

10.3.6.5
For TLP's, check of damage stability beyond what is described in [10.3.5.4] is not applicable.

10.4 Watertight integrity

10.4.1 General requirements

Material requirements to support watertight integrity are given in Sec.6.

10.4.2 External openings

Watertightness of external openings for evaluation of intact stability shall be assessed in accordance with DNVGL-OS-C301.

10.4.3 Internal openings

10.4.3.1 Internal openings need not be considered with respect to watertightness, unless damaged stability is opted for, see [10.1.1.5].

10.4.3.2 When damaged stability is opted for in accordance with [10.1.1.5], the number of openings in internal watertight subdivisions shall be kept to a minimum compatible with the design and proper operation of the floating structure. Wherever penetrations of watertight decks and bulkheads are necessary for access, piping, ventilation, and electrical cables; arrangements shall be made to maintain the watertight integrity of the enclosed compartments.

10.4.4 Capacity of watertight doors and hatch covers - operation and control

Reference is made to DNVGL-OS-C301.

10.4.5 Load lines

The hull shall be marked with load lines to allow for easy inspection of the draught and identification of any water ingress.

SECTION 11 CONTROL SYSTEM

11.1 Introduction

11.1.1 General

11.1.1.1 For the purposes of this section, the motion control system and wind turbine control system refer to the specific objectives of the control loop referred to and make no implication of the architectural separation or combination of each loop. The combination of all control systems is referred to as the combined control system. This section addresses the combined control system and the protection system for the wind turbine to the extent necessary for structural design.

Active control of coupled floater-turbine motions may be necessary to maintain operation within system design limits. It is possible that the motion control objective couples with the power control objective. The control loops dedicated to each objective shall therefore be designed and analysed in parallel or be demonstrated to be suitably decoupled.

The motion control system consists of a controller whose purpose is to explicitly stabilize and reduce the floating system motions and keep the structural responses at or below the levels assumed in design. It is also possible to have a stable floating system without explicit floater stabilization if the wind turbine control system is designed adequately. In this case the motions are not necessarily actively reduced, but they are also not excited.

Guidance note:

Motion control can be achieved by adjusting thrust forces on the rotor through manipulation of blade pitch and generator torque. For spar type floaters, the motion controller is typically used to control the excitation in the pitch mode of motion. For TLP type floaters, for which the pitch mode of motion is restrained, the motion controller is used to control the excitation in the surge mode of motion.

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11.1.1.2 It is a prerequisite that the wind turbine operation and safety are governed by a control and protection system as required by DNVGL-ST-0438.

11.1.1.3 The combined control system shall be demonstrated to be both safe and robust when applied to the combined wind turbine and floating system.

Guidance note:

Demonstrating the overall performance of the complete control system can be undertaken by means of analysis of the entire system in the time domain, e.g. by simulation. A time domain simulation, as opposed to frequency analysis for example, is more appropriate as it includes non-linearities. It is also important that the control system is modelled and simulated to behave as it would when installed, making sure the source code matches the one on the deployed system.

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11.1.1.4 Requirements for the combined controller are given in [11.2] to the extent necessary for structural design. Requirements for control systems specified in DNVGL-ST-0076 and DNVGL-ST-0438 for wind turbines apply also to the floater motion controller. These requirements include, but are not limited to, monitoring requirements.

11.1.1.5 Requirements for instrumentation and control systems are addressed in DNVGL-OS-D202 and safety shutdown systems in DNVGL-OS-A101. These offshore standards are rather generic, but their requirements may still be relevant for the motion control system. DNVGL-OS-D202 and DNVGL-OS-A101 have both been written mainly with a view to application for manned installations and should therefore be consulted with this in mind, in particular with respect to operator interface and maintenance issues. See also DNVGL-ST-0438.

11.1.1.6 For the general installation of instrumentation equipment and systems in a wind turbine, the reference in DNVGL-OS-D202 App.F, contains aspects that should be considered. DNVGL-OS-D202 applies to mobile offshore units and provides guidance that may be used for floating wind turbine structures only as far as applicable. Selected items from the DNVGL-OS-D202 contents list that may provide relevant guidance is presented in DNVGL-OS-D202 [F.3]. This guidance will in many cases have to be modified or adapted to the particular wind turbine design under consideration. See also DNVGL-ST-0438.

11.2 Floater-turbine coupled control

11.2.1 Background

11.2.1.1 Operation of the wind turbine at wind speeds above the rated wind speed implies that the wind turbine control system operates such that excess aerodynamic power above some threshold is discarded, for example through pitch angle control of the rotor blades. This applies to all wind turbines, regardless of whether they are floating or bottom-fixed. However, for a floating wind turbine in this wind speed region, the negative gradient of thrust with respect to wind speed and pitch angle can couple with the power control objective and introduce negative damping in the surge and pitch modes of motion. In these cases of negative damping, energy is added to the cyclic motion of the floating structure and will result in amplification of motion to a level that could exceed the limits of the structure.

11.2.1.2 The phenomenon described in [11.2.1.1] can be avoided through careful design of the combined controller. The design may use additional feedback signals from position or acceleration sensors, reflecting the floater motion, to calculate additional control actions for motion control or to prevent floater excitation. Actuators used for motion control can include but are not limited to

— blade pitch angle
— generator torque levels
— ballast water distribution.

11.2.2 Floater motion controller

11.2.2.1 A floater motion controller shall be implemented when intervention against amplification of motion and response is necessary to maintain operation within the design limits of the floating system.

The ability of the control system to meet system motion requirements shall be demonstrated by means of analysis of the entire floating system in the time domain, e.g. by simulation. The combined controller shall be tested in its entirety, and any separate control loops within the combined controller, such as motion control and wind turbine control cannot be separated for the purposes of this demonstration.

Guidance note:

In general, it is the purpose of the motion control to keep the extreme loads, the long-term fatigue stress histories, the motions and accelerations, the inclination and the deck clearance within the design envelope, and to keep the entire floating system stable.

In particular, to keep fatigue damage at a minimum, it is advisable to design the motion control system not only to keep the structural responses at or below the assumptions made for structural design, but also to limit the excitation of the floater as much as possible in relevant modes of motion such as pitch for a spar and surge for a TLP. For other types of floaters surge and roll may be relevant modes of motion.

The wind turbine control can excite the fore-aft bending of the tower considerably. This implies that even though a TLP is restrained in the pitch mode of motion, the pitch mode of motion may need to be considered also for this floater type.

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11.2.2.2 The motions of a floating support structure are influenced by the combined control. This may give rise to special load cases. Such special load cases shall be identified and shall be accounted for in structural design. These load cases include, but are not necessarily limited to

— severe sea states with controller failure

— failure in the coupling of the motion and turbine controllers when these are separate
— failure in the coupling of the motion and turbine controllers when these are integrated
— sensor failures.

11.2.2.3 The inclinations that might occur for floating wind turbine units may have an impact on systems such as the sensing systems and the activation systems. Such impact can, for example, be the effect of gravity and rotational motions on nacelle acceleration measurements and needs to be accounted for in analyses. Bandwidth requirements of inclinometers should be specified and accounted for when they are used in the control system. Sensors shall be appropriately designed to take into account the range of motions of the floater. The inclinations shall be less than the critical inclination which is the inclination that causes openings to become submerged or brings the deck below the water line.

11.2.2.4 Possible failures of the floater motion control system shall be considered. DNVGL-ST-0438 provides a list of such possible failure and addresses their effects.

11.3 Interaction with other systems

Caution must be exercised to ensure that sensitive systems, such as impressed current systems used to avoid damage from lightning, are reliable also when the floater motion controller is operating and that these systems remain in service without interruption under the conditions that prevail during this operation. See DNVGL-ST-0076.

SECTION 12 MECHANICAL AND ELECTRICAL SYSTEMS

12.1 Introduction

This section provides general requirements for various mechanical systems which are necessary for maintaining the normal operation of a floating wind turbine unit.

12.2 Mechanical systems

12.2.1 General

The possible impact of floater motions on the design of the wind turbine mechanical systems shall be considered. Such impact can be relevant for the gearbox, for some lubrication systems and for some hydraulic systems. Floater motions when the turbine is parked can be of importance for mechanical components such as bearings.

12.2.2 Bilge system

It shall be documented that bilge systems for removing and pumping bilge water serve the purpose. Bilge systems may be designed in accordance with requirements given in DNVGL-OS-D101.

12.2.3 Ballast system

12.2.3.1 Ballast systems can be powered by bilge systems or by compressed air systems. Ballast systems shall provide capability to ballast and deballast all ballast tanks except those that are used as permanent ballast tanks only. For other voids, such as ballast tanks that are used for permanent ballast only, it shall be assessed whether these voids need to be furnished with means to pump water, for example in the event of damaged voids.

12.2.3.2 Ballast systems can be designed in accordance with requirements given in DNVGL-OS-D101.

12.2.4 Compartment venting

12.2.4.1 Compartments and tanks that are equipped with fixed means of drainage shall also be equipped with a ventilation arrangement.

12.2.4.2 Sealed compartments may not need ventilation arrangement and may not need bilge systems. Passive ventilation can be provided via access opening such as doors and hatches.

12.2.5 Mooring equipment

Mooring equipment, such as winches and windlasses, chain stoppers, fairleads and systems to tension the mooring lines, can be designed in accordance with structural design procedures specified in DNVGL-OS-E301. The design of these components is then based on a load equal to the characteristic breaking strength of the mooring lines. In this context, DNVGL-OS-E301 provides information about the minimum breaking load and other aspects of importance for the design.

12.2.6 Cranes

12.2.6.1 Cranes may be relevant for several purposes, including pull-in of electrical cables, lifting supplies from service vessels, and lifting stretchers to service vessels. In particular, a davit hoist system above the boat landing may be useful. Cranes can be designed in accordance with DNVGL-ST-0378.

12.2.6.2 Cranes located in the nacelle and delivered together with the rotor−nacelle assembly might be designed for use on bottom-fixed wind turbines only. The use of such cranes on floating wind turbine units might pose some challenges and might require some operational restrictions to be imposed because of the motions of the floating units.

12.2.6.3 Special attention should be given to dynamic amplification factors for lifting of wind turbine components.

12.2.7 Installation systems and pull-in systems

Winches may be relevant for other purposes than mooring, for example winches for pull-in of cables. Winches for pull-in of cables may be portable winches which may be removed after the cable installation has been completed.

12.2.8 Turret systems

Turret systems consist of a large number of components. These components can be both mechanical components and electrical components. Depending on the actual turret system to be designed, an applicable standard shall be applied for the design.

12.2.9 Power generation systems

Power supply for winches and other equipment shall be installed.

12.2.10 Firefighting systems and equipment

Firefighting systems and equipment shall be designed under due consideration of the size, type and intended service of the floating wind turbine unit. DNVGL-OS-A101 provides a general description of the safety philosophy for firefighting systems and equipment.

12.2.11 Other equipment

Mechanical systems for tightening of tendons, for example following long-term creep and relaxation, shall be installed on floating units supported by tendons.

12.3 Electrical systems

12.3.1 Lightning and earthing system

A lightning and earthing system shall be in place. The lightning and earthing system shall be designed in accordance with the requirements of DNVGL-OS-D201 Ch.2 Sec.2 [9.6] and DNVGL-OS-D201 Ch.2 Sec.2 [9.7], and to the requirements of IEC 61892-6, clauses 4 and 16. Relevant parts of DNVGL-ST-0076 also apply.

SECTION 13 CORROSION PROTECTION

13.1 Introduction

13.1.1 General

The requirements for corrosion control given in DNVGL-RP-0416 apply to floating wind turbine structures with the exceptions, deviations and additions specified in this section.

13.1.2 Splash zone

13.1.2.1 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. The splash zone separates the atmospheric zone above and the submerged zone below.

13.1.2.2 For a floater, a different splash zone definition than for a bottom-fixed support structure applies and replaces the one given in DNVGL-RP-0416. The definition of the splash zone for external surfaces of a floating support structure is given in [13.1.2.3] and [13.1.2.4]. The definition of the splash zone for internal surfaces of a floating support structure is given in [13.1.2.5].

13.1.2.3 The upper limit of the external splash zone is the level on the floater corresponding to the highest still water level with a recurrence period of 1 year in combination with the deepest operational draught and increased by

— the crest height of a wave with height equal to the significant wave height with a return period of 1 year
— the foundation settlement, if applicable.

The lower limit of the external splash zone is the level on the floater corresponding to the lowest still water level with a recurrence period of 1 year in combination with the shallowest operational draught and reduced by

— the trough depth of a wave with height equal to the significant wave height with a return period of 1 year.

Guidance note:

For heave-restrained floaters, such as TLPs, the definition of the external splash zone becomes identical to the splash zone definition for bottom-fixed structures given in DNVGL-RP-416.

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13.1.2.4 When the draught can be adjusted to provide for satisfactory accessibility for inspection and repair, the lower limit of the external splash zone may be set equal to the upper limit of the splash zone, i.e. the external splash zone vanishes. It is a prerequisite for capitalizing on this option that an inspection and maintenance plan for the floater is in place with general visual inspections at regular intervals in accordance with specifications in DNVGL-ST-0126 Sec.9.

Guidance note:

Adjustment of the draught, e.g. by ballasting and deballasting, is possible for floaters which are not heave-restrained, and can be used as part of the maintenance strategy to avoid the requirements for coating and corrosion allowance in the region that would otherwise be defined as the external splash zone.

For heave-restrained floaters such as TLPs, such adjustment of the draught is not possible and the external splash zone has a defined extent and cannot be ignored.

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13.1.2.5 The upper limit of the internal splash zone is the highest operational ballast water level in the compartment in question. The lower limit of the internal splash zone is the lowest operational ballast water level in the compartment in question.

13.1.3 Corrosion allowance

13.1.3.1 Requirements for corrosion allowance in internal compartments, which are not corrosion protected by coating, are specified in DNVGL-RP-0416. In closed internal compartments which are permanently sealed by welding, these requirements for corrosion allowance may be waived. In other closed internal compartments, corrosion may be mitigated by control of humidity or level of oxygen and the requirements for corrosion allowance may be waived. For compartments containing solid ballast, it is a prerequisite for these waivers that the solid ballast does not contain sulphides.

Guidance note:

Corrosion control by exclusion of oxygen is primarily an option for structural compartments which are only externally exposed to seawater. 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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13.1.3.2 Corrosion allowance for chains used as mooring lines is defined as an increase in the chain diameter. Unless site-specific corrosion data for chains indicate otherwise, the corrosion allowance shall be taken as equal to or higher than the recommended minimum values for such corrosion allowance given in Table 13-1. Requirements for corrosion allowance specified by national authorities may in some cases be stricter than these recommended minimum values.

Table 13-1 Recommended minimum corrosion allowance for chain

Part of mooring line	*Corrosion allowance to be added to chain diameter (mm/year)*		
	Regular inspection [1)]	*Requirements for the Norwegian continental shelf*	*Requirements for tropical waters*
Splash zone	0.4	0.8[2)]	1.0
Catenary [3)]	0.3	0.2	0.3
Bottom [4)]	0.4	0.2	0.4

1) The regular inspection is carried out by ROV in accordance with DNVGL-RU-OU-0102 or in accordance with operator’s own inspection program, approved by national authorities if necessary.
2) The increased corrosion allowance in the splash zone is required by NORSOK M-001.
3) Suspended length of mooring line below the splash zone and always above the touchdown point.
4) The corrosion allowance specified in the table is provided as guidance. Significantly larger corrosion allowance than the minimum values recommended in the table should be considered if microbiologically induced corrosion (MIC) can be expected.

Guidance note:

The corrosion allowance given in Table 13-1 is based on values given in DNVGL-OS-E301.

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13.1.4 AC corrosion

The influence from energized power cables on corrosion and corrosion rates on the structural interface of the floater unit locally at the attachment point should be considered when the corrosion protection of this unit is designed.

Guidance note:

This recommendation reflects the potential for AC corrosion in a steel floater unit. An energized power cable can induce a current in the metallic floater unit and this current may accelerate the corrosion process locally at the attachment points.

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SECTION 14 TRANSPORT AND INSTALLATION

14.1 Marine operations

14.1.1 General

14.1.1.1 Unless otherwise specified, the requirements for planning and execution of marine operations for transport and installation given in DNVGL-ST-N001 and DNVGL-ST-0437 apply.

14.1.1.2 The installation of floating support structures and their station keeping systems in a large wind farm poses some logistic challenges and it is recommended to carry out a logistics study for the installation and to plan the logistics of the installation carefully. The need for inshore storage premises for temporary anchoring of the support structures shall be considered.

Guidance note:

The various structures and structural components in a wind farm cannot necessarily be installed on site directly from production, for example

— when they are mass-produced units rather than units tailor-made for the specific site
— when a weather window for the installation has to be awaited
— when the installation is dependent on preinstalled components such as preinstalled anchors,

for which reason inshore storage premises for intermediate anchoring or storage may be needed.

The seabed topography and the water depth, near shore as well as along the transportation route between the inshore storage site and the wind farm site, may be essential for how the transportation can actually be carried out and will pose site- and route-specific requirements for the maximum allowable draught and thereby for the floater stability in the intermediate transportation phase.

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14.1.1.3 DNVGL-RP-N103 provides guidance for modelling and analysis of marine operations.

14.2 Risk management during marine operations

14.2.1 Risk management in transportation and installation phases

DNVGL-RP-N101 provides guidance on how to manage and mitigate risks related to marine operations.

14.3 Marine warranty surveys

14.3.1 General

14.3.1.1 Marine warranty surveys are optional in the context of project certification of wind farms, but may be required by the insurance company in order to effect an insurance for temporary phases such as sea transport and installation.

14.3.1.2 For further details regarding marine warranty surveys of marine operations for transport and installation of structures and components for offshore wind farms see DNVGL-ST-N001.

14.4 Marine operations – general requirements

14.4.1 General

14.4.1.1 Requirements and recommendations relevant for marine operations during temporary phases are given in DNVGL-ST-N001 and are applicable to marine operations during temporary phases for floating wind turbine structures.

14.4.1.2 The requirements for vessel stability given in DNVGL-ST-N001 are applicable for vessels and barges used for transport and installation of floating wind turbine structures, but are not applicable for the floating wind turbine structures themselves.

14.4.1.3 Requirements for stability of floating wind turbine structures during transport and installation are given in Sec.10.

14.5 Marine operations – specific requirements

14.5.1 Load transfer operations

14.5.1.1 The load transfer operations cover load-out, float-out, lift-off and mating operations. For deep draught floaters such as spars, which may be floated to site in horizontal position, the load transfer operations also cover upending.

14.5.1.2 Turbine-structure mating can take place either inshore or on the wind farm site, i.e. either before or after the sea transport phase for the support structure. Turbine-structure mating on the wind farm site can take place either before or after the hook-up of the support structure to the station keeping system.

14.5.1.3 Requirements for load transfer operations are given in DNVGL-ST-N001.

14.5.2 Sea transports

14.5.2.1 Specific requirements and guidelines for single-vessel and barge-towing operations are given in DNVGL-ST-N001.

14.5.2.2 Requirements and recommendations for transport on-board ship, towing of multi-hull vessels, self-floating and self-propelled carrier transports are given in DNVGL-ST-N001.

14.5.2.3 Special considerations may be necessary in cases where the entire floating wind turbine unit, fully assembled, is to be transported from inshore premises to the wind farm site as opposed to the case where the floating support structure and the wind turbine are transported separately for subsequent mating on the wind farm site.

14.5.3 Offshore installation

14.5.3.1 Specific requirements and recommendations for offshore installation operations applicable for wind turbines and their support structures and structural components are given in DNVGL-ST-N001.

14.5.3.2 Environmental loads and load cases to be considered are described as well as on-bottom stability requirements and requirements for structural strength.

14.5.3.3 Operational aspects for ballasting and grouting shall be considered.

14.5.3.4 Operational aspects for mooring line installation, anchor installation and hook-up of mooring lines to support structure and anchors shall be considered.

14.5.4 Lifting operations

14.5.4.1 Guidance and recommendations for lifting operations, onshore, inshore and offshore, of objects with weight exceeding 50 tonnes are given in DNVGL-ST-N001.

14.5.4.2 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.

14.5.4.3 Turbine-structure mating on a deep-water wind farm site may pose some challenges in that crane operations for lifting of the turbine may have to be done from a floating crane vessel.

14.5.5 Subsea operations

Subsea operations are relevant for subsea components related to floating wind turbine installation, for example anchors and mooring lines. Subsea operations are also relevant for tie-in of, for example, electrical cables. Planning, design and operational aspects for installation of such components are described in DNVGL-ST-N001.

SECTION 15 IN-SERVICE INSPECTION, MAINTENANCE AND MONITORING

15.1 Introduction

15.1.1 General

15.1.1.1 The provisions set forth in DNVGL-ST-0126 for in-service inspection, maintenance and monitoring shall apply.

15.1.1.2 An inspection interval of at most 5 years applies when design fatigue factors (DFFs) specified in Sec.7 for structural components, which are accessible for inspection, are used in the design of such components. Longer inspection intervals can be applied in design; however, this will require other requirements for DFF to be established than those specified in Sec.7.

15.1.1.3 For corroded steel structures, inspections shall include measurements of plate thicknesses by ultrasonic testing to document the degradation.

15.1.2 Anchors, mooring chain and steel tendons

The provisions set forth in DNVGL-OS-E301 for in-service inspection, maintenance and monitoring shall apply.

15.1.3 Fibre ropes, tethers and tendons made from synthetic fibre yarns

15.1.3.1 The provisions set forth in DNVGL-OS-E303 for in-service inspection, maintenance and monitoring shall apply.

15.1.3.2 The in-service condition management program shall be based on tension monitoring and control of temperature in order to manage the 3-T margins throughout the design life.

SECTION 16 POWER CABLE DESIGN

16.1 Introduction

16.1.1 General

16.1.1.1 This section gives criteria, requirements and guidance for structural design and analysis of power cable systems exposed to dynamic loading for use in the floating wind industry.

16.1.1.2 This section refers to a number of available design codes of relevance for power cable design. Where requirements specified in this section conflict with those of referenced standards, the provisions of this section shall prevail.

16.1.1.3 Wherever normative references to other codes are made, the provisions of these other codes shall be adhered to in design.

16.1.1.4 The design and qualification of subsea cables considering electrical, functional and environmental aspects shall be in accordance with the requirements given in DNVGL-ST-0359.

16.2 Overview of relevant standards

16.2.1

An overview of international recognized design codes addressing design issues of relevance for submarine power cables is given in Table 16-1. The codes listed in Table 16-1 are relevant for dynamic power cables and their design, including the cable-floater interface.

16.2.2

Although several of the codes listed in Table 16-1 refer specifically to flexible pipes and umbilicals, the principles and methodology applied in the codes may for the most part be applied to subsea power cables. The main shortcoming is the lack of acceptance criteria, i.e. allowable strain or utilization of yield, of the various materials and components of a power cable.

Table 16-1 Relevant documents for design, analysis and testing of dynamic power cables

Design aspects	*Document code*
General requirements for subsea power cable installations covering the full life-cycle	DNVGL-ST-0359
Guidance for all phases of the life cycle of subsea power cable projects, with a focus on static service in shallow water renewable energy applications	DNVGL-RP-0360
Main reference for mechanical design of dynamic umbilicals, providing requirements for global load effect analyses and requirements for local load effect analyses	ISO 13628-5
General requirements for marine operations, including installation of submarine power cables	DNVGL-ST-N001
Analysis guidance: — outline of global response model verification — guidance on statistical response processing.	DNVGL-OS-F201

Design aspects	*Document code*
Fatigue design capacity curves of standard materials	DNVGL-RP-C203
Specification of environmental loading, choice of hydrodynamic coefficients etc., and principles for floater motion analysis	DNVGL-RP-C205
Analysis methodology for calculating maximum allowable free span length	DNVGL-RP-F105
Recommendations for a risk-based approach to pipeline protection that may also be applied to subsea power cables	DNVGL-RP-F107
Analysis methodology for checking stability on the seabed and for calculating minimum required submerged weight to achieve stability	DNVGL-RP-F109
Principles for assessment of riser interference	DNVGL-RP-F203
Principles for riser fatigue assessment and simplified VIV analysis guidance	DNVGL-RP-F204
Guidance on floater motion and station-keeping analysis	DNVGL-RP-F205
Supplement to ISO 13628-5, covering subsea power cables: — design and acceptance criteria for power cables, cable components and cable terminations.	DNVGL-RP-F401
Acceptance criteria for tensile armour: — acceptance criteria for polymer layers in flexible pipes.	API Spec. 17J
Design guidance for ancillary components such as buoyancy modules, bend stiffeners etc.	API Spec. 17J

16.3 Design principles

16.3.1 Basic principles

16.3.1.1 The power cable system, including its interface with the floater, shall be designed in accordance with the following basic principles:

- the power cable system shall satisfy functional and operational requirements as given in the design basis
- the power cable system shall be designed such that an unintended event does not escalate into an accident of significantly greater extent than the original event
- the power cable system shall permit simple and reliable installation, retrieval, and be robust with respect to use
- the power cable system shall provide adequate access for inspection, maintenance, replacement and repair
- design of structural details and use of materials shall be done with the objective to minimize the effect of corrosion, erosion and wear
- acceptance criteria, ensuring the structural integrity of the power cable, shall be defined for all cable components.

16.3.1.2 The power cable system shall be designed, manufactured, installed and operated in such a way that:

- with acceptable probability, it will remain fit for the use for which it is intended, with due regard to its service life and cost, and
- with an appropriate degree of reliability, it will sustain all foreseeable load effects and other influences likely to occur during installation and service.

16.3.1.3 The power cable system shall be designed in accordance with relevant IEC standards, as described in DNVGL-RP-0360, and shall comply with requirements for mechanical analysis and strength, as specified herein.

16.3.1.4 A design basis, containing or referring all relevant requirements to the power cable system, shall be established, as described in DNVGL-ST-0359.

16.3.1.5 A design methodology including, as a minimum, a description of analysis methods and assumptions made, as well as acceptance criteria, shall be established and presented as part of the design documentation for the power cable system.

16.3.2 Design approach

16.3.2.1 The power cable with structural end terminations, such as hang-offs, shall be designed with respect to relevant load cases related to both installation and operation (i.e. in service).

Guidance note:

Design and fabrication of a power cable system is often initiated, and sometimes even completed, before detailed planning of the installation operation is performed. Input from relevant installation analyses is therefore often unavailable in the design phase of a cable system. However, as some of the load cases related to installation may turn out to be governing with respect to design of the power cable and/or its end terminations, conservative estimates of installation loads should be applied at the design stage of the power cable system in order to ensure that it is installable.

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16.3.2.2 The partial safety factor method is often applied in design of steel structures such as cable terminations and other cable accessories. 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 material factors to characteristic reference values of the basic variables (see Sec.2 for further explanation). The basic variables are, in this context, defined as:

— loads acting on the structure or their load effects in the structure, and
— resistance of the structure or resistance of materials in the structure.

16.3.2.3 A working stressdesign/allowable stress design (WSD/ASD) is typically applied in design of power cables. This design format expresses the structural safety margin by one single safety factor, or utilization factor, for each limit state. The WSD method adopted herein applies explicit design checks similar to those of the partial safety factor method, but accounts for the influence of natural variability and other uncertainty in only one single utilization factor. See DNVGL-OS-F201 for further explanation.

Guidance note:

The utilization factor, η, accounts for the integrated uncertainty and possible bias in load effects and resistance, and may be interpreted as an inverted weighted product of partial safety factors. In other codes, the utilization factor is also known as an allowable stress factor or design factor.

The WSD approach enables the use of capacity curves that define a cable's capacities with respect to combined tension and curvature for different utilization factors. See ISO 13628-5 for further information.

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16.3.2.4 The power cable system, i.e. power cable and accessories, shall be checked for the following limit states during both installation and operation:

— ULS
— ALS, as applicable
— FLS.

For details about these limit states and how they are defined, see Sec.1 and Sec.2.

16.4 Functional requirements

16.4.1 Power cable

The power cable shall, at a minimum, be designed to:

— meet the power transmission requirements specified by the client
— enable installation by a vessel with installation equipment characteristics as specified by the client
— operate at the specified effect, within the specified ambient temperature range
— enable one recovery and reinstallation, as a minimum, to account for unforeseen events during the installation operation
— ensure corrosion protection of the power cable components throughout the specified service life of the power cable system
— function as intended for the duration of its intended service life.

16.4.2 Pull-in arrangement

16.4.2.1 A pull-in arrangement, such as a pull-in head, “Chinese fingers” or similar, shall be used to pull the cable end up on the floater or onshore.

16.4.2.2 The pull-in arrangement shall ensure a safe transfer of cable tension to the pull-in wire without jeopardizing the functionality of the power cable and accessory equipment such as hang-off termination, floater connector, bend stiffener or similar.

16.4.3 Hang-off termination

The power cable shall be terminated with a hang-off termination at the floater end, designed to:

— transfer loads from the suspended power cable to the floater structure
— ensure corrosion protection of the power cable and accessory equipment at the floater interface throughout the specified service life of the power cable
— enable a safe and efficient installation
— function as intended for the duration of its specified service life.

16.4.4 Bend stiffener/bell-mouth

16.4.4.1 A bend stiffener or bell-mouth at the floater interface may be applied to reduce local bending stresses in the cable components and distribute fatigue damage over the length of the bend stiffener.

16.4.4.2 A bend stiffener/bell-mouth shall be designed:

— to limit bending stresses in the power cable
— to ensure that the cable meets the specified fatigue design criterion
— to ensure that the functionality of the power cable is maintained throughout its specified service life
— for all relevant combinations of associated cable tension and angle at the floater interface, resulting from floater motions
— considering maximum cable and bend stiffener (if relevant) temperature, accounting for the increase in cable temperature resulting from operation at full effect, including the insulating effect of the bend stiffener material
— considering minimum cable and bend stiffener (if relevant) temperature, accounting for the worst-case condition where a shut-down of power transmission coincides with the anticipated minimum temperature over the service life of the power cable system.

16.4.5 Floater structural interface

16.4.5.1 A connector arrangement, consisting of one part pre-installed on the floater and another part assembled to the floater end of the cable, may be applied to simplify connection of a cable or its bend stiffener at the floater interface.

16.4.5.2 The connector arrangement shall be considered as part of the floater structure and shall be designed in accordance with Sec.7.

16.4.5.3 The connector arrangement shall be designed with respect to anticipated loads from the suspended cable and bend stiffener, if applicable, resulting from floater motions throughout the specified service life.

16.4.5.4 The connector part assembled to the power cable during installation shall be designed for relevant installation loads in accordance with [16.6].

16.5 Analysis methodology

16.5.1 Power cable mechanical properties

As a minimum, the following properties shall be specified by the cable supplier:

— unit mass in air
— unit weight, submerged (interstices empty)
— unit weight, submerged (flooded)
— bending stiffness
— axial stiffness
— torsional stiffness
— torque balance factor (angle of twist vs. tension).
— If axial compression is allowed, the cable supplier shall also specify the torque balance factor and the axial and bending stiffness in compression, as these properties may differ substantially in tension and compression. Nonlinear stiffness properties in tension and compression shall be considered in the dynamic analyses of the cable installation and the operational extreme analyses.

16.5.2 Power cable structural capacities

16.5.2.1 Accounting for design resistance, addressed in [16.7], the following shall be established (as a minimum):

— minimum allowable storage radius
— minimum allowable bending radius at low tension during handling and installation, i.e. load-controlled condition (see guidance note below)
— minimum allowable bending radius at maximum anticipated installation tension, i.e. load-controlled condition (see guidance note below)
— maximum allowable tension at bending radius corresponding to the radius of the installation chute, an I-tube bend or other, i.e. displacement-controlled condition (see guidance note below)
— maximum allowable axial compression and corresponding minimum bending radius
— maximum allowable tensioner squeeze load for 2-track, 3-track and/or 4-track tensioner, as applicable
— maximum allowable twist at zero and maximum installation tension.

Guidance note:

A load-controlled condition designates a condition in which magnitude and direction of bending are governed by product loads only, while a displacement-controlled condition designates a condition in which product bending is physically constrained, e.g. over chute, radius controller or similar, or around an I-tube bend.

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16.5.2.2 For displacement-controlled bending, cable ovalization and deformation of the cable internals shall be considered.

16.5.2.3 In order to allow axial compression, the following shall be documented by the product supplier:

— an FMEA to identify both global (operational) and local (internal) failure modes and effects
— structural integrity and functionality of the product for the most onerous combination of axial compression and curvature considering cyclic effects
— minimum safety factor, accounting for manufacturing tolerances and uncertainties

If not specified, maximum allowable compression shall be taken as zero.

16.5.3 Installation analyses

16.5.3.1 All phases of the cable installation operation shall be analysed in accordance with the principles stated in DNVGL-ST-N001 Sec.7.

16.5.3.2 The installation analyses shall demonstrate that the power cable system may be installed using the intended vessel and installation equipment without risk of damage to personnel, product or equipment.

Guidance note:

Principles for operational planning are presented in DNVGL-ST-N001, together with definitions of unrestricted and weather restricted operations. Weather restricted is defined to be when the duration of operation, including contingency, is less than 72 hours. Note that even if the total operation lasts more than 72 hours, the operation may be defined and designed as weather restricted, given that the cable lay operation is planned and performed based on 'running' weather windows, while ensuring that the operation can be suspended and the cable be brought into a safe condition if the weather should deteriorate beyond the lay criteria.

For further details, see DNVGL-ST-N001.

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16.5.3.3 The cable installation analyses shall establish limiting weather conditions for the installation operation, as well as relevant installation parameters (e.g. maximum lay tension, minimum lay angle, minimum lay-back etc.) required in order to carry out the installation operation in a safe manner and within the structural capacities of the cable, see [16.5.2].

16.5.3.4 Unless more detailed evaluations of current velocity are made, the characteristic current shall be taken as the 10-year return value.

Guidance note:

For short operations, i.e. weather restricted, it could be applicable to define a maximum operation limiting current velocity. In this case, current prediction and monitoring during operations are necessary in order to ensure that the limiting current velocity is not exceeded.

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16.5.4 Global (in-service) analyses

16.5.4.1 For permanent operational conditions, a 50-year return period ($2 \cdot 10^{-2}$ annual exceedance probability) applies for definition of characteristic environmental loads. Analysis methodology specified in

DNVGL-OS-F201 may be applied as long as this definition of characteristic environmental loads is adhered to and as long as safety factors specified in [16.6] and [16.7]are used.

16.5.4.2 Global load effect analyses of the power cable system shall be performed in accordance with DNVGL-OS-F201.

16.5.4.3 Fatigue analysis of the power cable system shall consider all relevant cyclic load effects, as described in DNVGL-OS-F201, including:

— first order wave effects (direct wave loads and associated floater motions)
— second order floater motions
— thermal stress cycles, e.g. due to electrical power loading
— vortex induced vibrations (VIV).

16.5.4.4 The effect of VIV on the suspended power cable shall be considered.

16.5.4.5 Interference with mooring lines and any neighboring cables shall be considered. The methodology in DNVGL-RP-F203 is recommended.

16.5.4.6 Accurate modelling of the following shall be ensured in all dynamic analyses:

— cable mass and submerged weight considering filling of interstices as appropriate
— marine growth
— the cable's stiffness properties considering temperature
— bend stiffener properties considering temperature
— uplift of buoyancy elements accounting for manufacturing tolerances and loss of buoyancy due to water ingress, creep etc.

16.5.4.7 The estimation of hydrodynamic load on power cables subjected to accumulated marine growth shall account for the increase in effective diameter and surface roughness. DNVGL-RP-C205 may be used for guidance.

16.5.4.8 It shall be ensured that the system configuration is robust, i.e. that cable capacity is not exceeded because of an underestimation of critical input parameters. Robustness may be ensured by assuming conservative values for all critical parameters in the analyses. Alternatively, nominal or expected values may be assumed in the analyses and sensitivity checked separately. Sensitivity analyses shall be performed to

— identify parameters that may have a significant effect on the results (i.e. critical parameters) and/or
— verify that the input parameters applied in the analyses are reasonably conservative (i.e. not overly conservative).

16.5.4.9 For the sensitivity analyses required in [16.5.4.8], the following critical parameters should be considered, as applicable:

— combination of wind-driven waves and swell from different directions
— drag and inertia effects
— structural damping
— friction coefficients between cable internals
— seabed stiffness and friction
— current
— floater draught and corresponding motions (i.e. transfer functions)
— cable installation tolerances (e.g. location of cable touchdown point etc.).

16.5.4.10 Engineering judgment based on previous experience with similar load cases may be used to identify critical parameters. The engineering judgment shall be justified through reference to relevant experience, if applicable. Justification of the values assumed for critical parameters shall also be given. If relevant limit state criteria are not satisfied in a sensitivity check, better control of the relevant parameter or assumption should be ensured, e.g. by introducing suitable limitations in the operational procedures or by monitoring.

16.6 Loads and load effects

16.6.1 Characteristic loads

16.6.1.1 Functional, environmental and accidental loads are defined in Sec.4.

16.6.1.2 Characteristic loads and load effects are defined in Sec.2 and Sec.4. These definitions apply both to the power cable and to the cable accessories.

16.6.1.3 For permanent (operational) conditions, a 50-year return period ($2 \cdot 10^{-2}$ annual exceedance probability) applies for definition of characteristic environmental loads.

16.6.1.4 For temporary conditions the load effect return period for definition of characteristic environmental load effects depends on the seasonal timing and duration of the temporary period. The return periods shall be defined such that the probability of exceedance in the temporary state is no greater than that of the long-term operational state. For details about definition of characteristic values for environmental loads and for loads of other categories, see Table 4-1 and Table 4-2.

16.6.1.5 Characteristic load effects in the power cable and accessories under temporary (installation) conditions shall be determined in accordance with DNVGL-ST-N001.

16.6.2 Design load effects for power cables

16.6.2.1 An ASD/WSD approach is typically applied in design of subsea power cables. The design load effect S_d for a power cable is therefore equal to the sum of unfactored characteristic load effect contributions from functional loads, environmental loads and/or accidental loads, as applicable.

16.6.2.2 Design load effects for short-term (e.g. cable installation) conditions shall be determined in accordance with DNVGL-ST-N001.

16.6.2.3 Design load effects for long-term (i.e. in service) conditions, shall be determined in accordance with the principles of DNVGL-OS-F201, using characteristic loads as specified in Table 4-2 and applying the load and material factors specified in Sec.5.

16.6.3 Design load effects for cable accessories of steel

16.6.3.1 A partial safety factor approach is typically applied in design of cable accessories of steel. Design load effects are therefore the sum of factored characteristic load effect contributions from functional loads, environmental loads and/or accidental loads, as applicable:

$$S_d = (S_{FC} \cdot \gamma_{Fd} + S_{EC} \cdot \gamma_{Ed} + S_{AC} \cdot \gamma_{Ad}) \cdot \gamma_C$$

where

S_d	=	design load effect
S_{FC}, S_{EC}, S_{AC}	=	characteristic functional, environmental and accidental load effects, respectively

$\gamma_{Fd}, \gamma_{Ed}, \gamma_{Ad}$	=	functional, environmental and accidental load factors, respectively
γ_C	=	consequence factor applicable to lifting operations, see [6.3.3] and DNVGL-ST-N001.

16.6.3.2 Load factors applicable to short-term and long-term conditions shall be selected in accordance with DNVGL-ST-N001 and Table 5-1, respectively. For cable accessories not considered as lifting equipment, see 6.3.3, the consequence factor shall be set to 1.0.

16.6.3.3 Pull-in heads, cable terminations and other accessories supporting the weight of a suspended cable during installation, shall be regarded as lifting equipment and designed temporary (installation) conditions, applying a consequence factor in accordance with DNVGL-ST-N001 Sec.16.

16.6.3.4 For well planned, one time pull-in operations to an onshore or offshore facility, the consequence factor may be reduced or discarded, providing the following conditions are satisfied:

- a risk assessment of the operation has been carried out to evaluate potential consequences of winch and winch wire failure, and
- it can be documented that a failure of the end termination/fitting, pull-in head or installation aid will not cause personnel injury, environmental pollution or damage to vessel, installation equipment or other installations, and
- necessary contingency procedures are in place, supported by analyses, as appropriate.

16.6.3.5 The skew load factor specified in DNVGL-ST-N001 applies to lifts using several slings, and is therefore not relevant with respect to design of pull-in heads. The skew load factor is meant to account for a possible load increase in a sling due to differences in sling lengths caused by manufacturing tolerances. See DNVGL-ST-N001 for further guidance related to out-of-plane loads. In design of pull-in heads and pad-eyes, maximum wire angle relative to the pull-in head longitudinal axis shall be considered.

16.7 Resistance

16.7.1 Characteristic resistance

16.7.1.1 Material strength for cable components of structural steel is represented by SMYS and SMTS. The characteristic strength of structural steel shall be taken as the smaller of:

- SMYS adjusted for temperature derating
- 90% of SMTS adjusted for temperature derating.

16.7.1.2 The characteristic resistance R_k is calculated from the characteristic strength and the nominal properties of the structural component in question. This applies to the power cable as well as to the cable accessories.

16.7.1.3 The possible influence of temperature on material strength shall be accounted for, considering both environmental and operational (electrical) effects.

Guidance note:

Details of stress derating of steel owing to temperature effects are given in DNVGL-OS-F201.

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16.7.2 Design resistance

16.7.2.1 The design resistance of load-bearing cable components of steel (compliant with an ASD/WSD approach) is given as:

$$R_d = \eta \cdot R_k$$

where

R_d = design resistance of the steel component
η = utilization factor relevant for the given capacity check
R_k = characteristic resistance of the steel component.

16.7.2.2 The design resistance of cable accessories of steel (compliant with an LRFD approach) is given as:

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

where

R_k = characteristic resistance of the structural component
γ_R = resistance factor (or a combination of factors) relevant for the given capacity check.

16.7.3 Utilization factors and resistance factors

16.7.3.1 For design of power cables, the utilization factors for load-bearing cable components of steel shall not be taken greater than the values specified in Table 16-2.

Table 16-2 Utilization factors for steel armour

Load-bearing cable components of steel	*Utilization factor, η*
ULS for normal loads, consequence class 1	0.67
Installation, Consequence class 1	0.67
ALS for accidental loads	1.00

16.7.3.2 For design of cable accessories of steel with respect to installation conditions, the material factor shall not be taken less than γ_m = 1.15 in the ULS and the ALS.

16.7.3.3 For design of cable accessories in the operational condition, the material factor shall not be taken less than γ_m = 1.10 in the ULS and the ALS.

16.8 Design checks

16.8.1 Ultimate and accidental limit states

The power cable and accessories shall be checked against the ULS and ALS under short-term (installation) and long-term (operational) conditions. It shall be demonstrated that the following criterion is satisfied

$$S_d \leq R_d$$

where

S_d = design load effect, see [16.6.2] and [16.6.3]

R_d = design resistance, see [16.7.2].

16.8.2 Fatigue limit state

16.8.2.1 The power cable shall be qualified with respect to fatigue through testing in accordance with DNVGL-RP-F401 App.A.

16.8.2.2 Armour terminations subjected to significant fatigue loading, such as hang-off terminations, shall be qualified with respect to fatigue by testing.

16.8.2.3 Effects of friction and stick/slip behaviour between layers in the cable cross-section shall be considered in calculations of fatigue damage.

16.8.2.4 The design cumulative fatigue damage is

$$D_D = DFF \cdot D_C$$

in which D_C denotes the characteristic cumulative fatigue damage, e.g. calculated as outlined in Sec.7, and *DFF* is the design fatigue factor.

16.8.2.5 For steel components, *DFF* shall not be taken less than 10 unless specified otherwise.

16.8.2.6 The design criterion in fatigue is

$$D_D \leq 1.0.$$

16.9 Other issues

16.9.1 Interaction with fishing equipment

Possible interaction with fishing equipment, such as trawls, shall be considered in design.

16.9.2 On-bottom stability

On-bottom stability of power cables resting on the seabed shall be addressed in the cable design. DNVGL-RP-F109 can be used for on-bottom stability design of power cables resting on the seabed.

16.9.3 Free-span analysis

The principles of DNVGL-RP-F105 may be applied in analyses of free-spanning power cables.

16.9.4 Corrosion

16.9.4.1 The material in the water barrier sheath of the power cable shall be chosen such that it has sufficient resistance to corrosion considering the service environment: exposure to sea water, temperature. There shall be no penetration of the sheath due to corrosion (in terms of holes, pits, cracks etc.) during the service life of the cable.

16.9.4.2 Electrical continuity or electrical insulation, as required, between the cable components, hang-off termination and floater structure shall be ensured considering the overall system design.

16.9.5 Earthing of lightning conductors

Earthing of lightning conductors between turbine tower and floater and between floater and seabed shall meet the requirements for lightning and earthing system specified in Sec.12.

16.9.6 Protection against mechanical damage

It is recommended to carry out a risk analysis to assess the need for protection of power cables against mechanical damage, e.g. from dropped objects. See DNVGL-RP-F107, which provides guidance on which types of protection are suitable for different purposes.

16.9.7 Redundancy

It is recommended to consider cable redundancy in design of power cables and cable system layouts.

Guidance note:

Keywords for redundancy in case of cable failure are alternative routes for running power and more than one export cable. In this context, optimization should be kept in mind by balancing cost and regularity.

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CHANGES - HISTORIC

There are currently no historical changes for this document.

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