OFFSHORE STANDARD

DNV-OS-J103

Design of Floating Wind Turbine Structures

JUNE 2013

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FOREWORD

DNV is a global provider of knowledge for managing risk. Today, safe and responsible business conduct is both a license to operate and a competitive advantage. Our core competence is to identify, assess, and advise on risk management. From our leading position in certification, classification, verification, and training, we develop and apply standards and best practices. This helps our customers safely and responsibly improve their business performance. DNV is an independent organisation with dedicated risk professionals in more than 100 countries, with the purpose of safeguarding life, property and the environment.

DNV service documents consist of among others the following types of documents:

— *Service Specifications.* Procedural requirements.

— *Standards.* Technical requirements.

— *Recommended Practices.* Guidance.

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

A) Qualification, Quality and Safety Methodology

B) Materials Technology

C) Structures

D) Systems

E) Special Facilities

F) Pipelines and Risers

G) Asset Operation

H) Marine Operations

J) Cleaner Energy

O) Subsea Systems

U) Unconventional Oil & Gas

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

CHANGES – CURRENT

General

This is a new document.

Acknowledgments

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— Statoil ASA, Norway
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SECTION 1 INTRODUCTION

1 Introduction

1.1 General

1.1.1 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.

1.1.2 The standard covers structural design of floating wind turbine structures. The standard takes transportation, installation and inspection issues into account to the extent necessary in the context of structural design. The design principles and overall requirements are defined in the standard. Wherever possible, the standard makes reference to requirements set forth in its fixed-structures counterpart, DNV-OS-J101.

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

1.1.4 The standard does not cover design of wind turbine components such as nacelle, rotor, generator and gearbox. For structural design of rotor blades DNV-DS-J102 applies. For structural design of wind turbine components for which no DNV standard exists, the IEC 61400-1 standard applies.

1.1.5 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.

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 many floating structures it may be impractical to maintain the tower stiffness, since the turbine type approval that 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.

The important issue in this context is whether or not the turbine meets its performance and safety requirements as assumed for the type approval. A change in mass and stiffness distributions over the height of the tower will imply altered eigenfrequencies of the tower. Higher-order tower bending eigenfrequencies are coupled to the rotor loads and may, if changed, lead to undesirable vibrations such as whirling.

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1.1.6 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.

1.1.7 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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1.1.8 To ensure that structures designed according to this standard are manufactured in compliance with assumptions made during design, adequate follow-up of the manufacturing shall be carried out by an independent third-party surveyor in the manufacturing phase.

1.2 Objectives

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

1.2.2 The objectives of this standard are to:

- provide an internationally acceptable level of safety by defining minimum requirements for structures and structural components (in combination with referenced standards, recommended practices, guidelines, etc.)
- serve as a contractual reference document between suppliers and purchasers related to design, construction, installation and in-service inspection
- serve as a guideline for designers, suppliers, purchasers and regulators

— specify procedures and requirements for floating wind turbine structures subject to DNV certification
— serve as a basis for verification of floating wind turbine structures for which DNV is contracted to perform the verification.

1.3 Scope and application

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

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

1.3.3 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.

1.4 Non-DNV codes

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

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

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

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

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

1.5 Equivalence and future developments

1.5.1 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.

Guidance note:

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

Risk acceptance criteria may be taken according to IMO MSC/Circ.1023–MEPC/Circ.392 “Guidelines for Formal Safety Assessment”.

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1.5.2 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.

1.6 Hierarchy of DNV Offshore Publications

1.6.1 DNV Offshore Publications are organized according to a three-level document hierarchy, with these main features:

— Principles and procedures related to DNV's certification and verification services are separate from technical requirements and are presented in DNV Offshore Service Specifications
— Technical requirements are issued as self-contained DNV Offshore Standards
— Associated product documents are issued as DNV Recommended Practices.

1.6.2 The hierarchy in [1.6.1] is designed with these objectives:

— Offshore Service Specifications present the scope and extent of DNV's services
— Offshore Standards are issued as neutral technical standards to enable their use by national authorities, as international codes and as company or project specifications without reference to DNV's services
— The Recommended Practices provide DNV's interpretation of safe engineering practice for general use by industry.

Guidance note:

The latest revision of all DNV documents may be found in the list of publications on the DNV web site www.dnv.com.

New DNV documents, referenced in this standard, but not yet in effect by May 2013, are indicated by *italics*.

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2 References

2.1 General

2.1.1 The DNV documents listed in Table 1-1 and Table 1-2 and the recognized codes and standards in Table 1-3 are referred to in this standard.

2.1.2 The latest valid revision of each of the DNV reference documents in Table 1-1 and Table 1-2 applies.

Table 1-1 DNV Standards, Offshore Standards, Rules for Classification and Rules for Certification

Reference	*Title*
DNV-OS-A101	Safety Principles and Arrangements
DNV-OS-B101	Metallic Materials
DNV-OS-C101	Design of Offshore Steel Structures, General (LRFD Method)
DNV-OS-C103	Structural Design of Column Stabilised Units (LRFD Method)
DNV-OS-C105	Structural Design of TLPs (LRFD Method)
DNV-OS-C106	Structural Design of Deep Draught Floating Units/Spars (LRFD and WSD Method)
DNV-OS-C301	Stability and Watertight Integrity
DNV-OS-C401	Fabrication and Testing of Offshore Structures
DNV-OS-C501	Composite Components
DNV-OS-C502	Concrete Structures
DNV-OS-D101	Marine and Machinery Systems and Equipment
DNV-OS-D201	Electrical Installations
DNV-OS-D202	Automation, Safety, and Telecommunication Systems
DNV-OS-E301	Position Mooring
DNV-OS-E302	Offshore Mooring Chain
DNV-OS-E303	Offshore Fibre Ropes
DNV-OS-E304	Offshore Mooring Steel Wire Ropes
DNV-OS-F201	Dynamic Risers
DNV-OS-H101	Marine Operations, General
DNV-OS-H102	Marine Operations, Design and Fabrication
DNV-OS-H201	Load Transfer Operations
DNV-OS-H205	Lifting Operations
DNV-OS-J101	Design of Offshore Wind Turbine Structures
DNV-DS-J102	Design and Manufacture of Wind Turbine Blades
	Rules for Planning and Execution of Marine Operations
Standard for Certification No. 2.22	Lifting Appliances

Table 1-2 DNV Recommended Practices and Classification Notes	
Reference	*Title*
DNV-RP-A203	Qualification of New Technology
DNV-RP-C103	Structural Design Of Column Stabilised Units (LRFD Method)
DNV-RP-C104	Self-Elevating Units
DNV-RP-C201	Buckling Strength of Plated Structures
DNV-RP-C202	Buckling Strength of Shells
DNV-RP-C203	Fatigue Design of Offshore Steel Structures
DNV-RP-C205	Environmental Conditions and Environmental Loads
DNV-RP-C207	Statistical Representation of Soil Data
DNV-RP-E301	Design and Installation of Fluke Anchors in Clay
DNV-RP-E302	Design and Installation of Plate Anchors in Clay
DNV-RP-E303	Geotechnical Design and Installation of Suction Anchors in Clay
DNV-RP-E304	Condition Management of Offshore Fibre Ropes
DNV-RP-E305	Design, Testing and Analysis of Offshore Fibre Ropes
DNV-RP-F107	Risk Assessment of Pipeline Protection
DNV-RP-F109	On-Bottom Stability Design of Submarine Pipelines
DNV-RP-F203	Riser Interference
DNV-RP-F204	Riser Fatigue
DNV-RP-F205	Global Performance Analysis of Deepwater Floating Structures
DNV-RP-F401	Electrical Power Cables in Subsea Applications
DNV-RP-H101	Risk Management in Marine and Subsea Operations
DNV-RP-H103	Modelling and Analysis of Marine Operations
Classification Notes No. 20.1	Stability documentation for approval
Classification Notes No. 30.4	Foundations
Classification Notes No. 30.7	Fatigue Assessment of Ship Structures

Table 1-3 Other references	
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
DNV-OSS-102	Rules for Classification of Floating Production, Storage and Loading Units
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 1993-1-8	Eurocode 3: Design of steel structures – Part 8: Design of joints
EN 1997-1	Eurocode 7: Geotechnical Design – Part 1: General rules
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

3 Definitions

3.1 Verbal forms

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

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

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

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

3.1.5 *Agreement, or by agreement:* Unless otherwise indicated, agreed in writing between contractor and purchaser.

3.2 Terms

3.2.1 *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 nonlinear environmental conditions.

3.2.2 *Articulated tower:* An articulated tower 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.

3.2.3 *Bell-mouth:* Tapered section at the end of a pipe, often at the outlet of an I-tube or a J-tube.

3.2.4 *Bilge system*: System for pumping and removing bilge water.

3.2.5 *Chinese fingers:* Cable grip (stocking), made up of braided wire rope, used to pull or support cable tension.

3.2.6 *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.

3.2.7 *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.

3.2.8 *Compliant tower:* A compliant tower is a 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 they are generally well outside wave periods.

3.2.9 *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 transfers cable loads to the floater structure throughout the specified service life of the power cable system.

3.2.10 *Deep Draught Floater (DDF):* Spar or similar type platform with a relatively large draught compared to ship-shaped floaters and semisubmersibles.

3.2.11 *Deep water:* Deep waters are characterized by wavelengths shorter than about twice the water depth.

3.2.12 *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.

3.2.13 *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 reckoned 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.

3.2.14 *Heel:* A tilt, as of a boat, to one side; used here in the context of wind heeling when floating stability is the issue.

3.2.15 *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.

3.2.16 *Lightweight:* The invariable weight of the floating structure; i.e. the basis for calculating the loading conditions for evaluation of floating stability. Anchors and cables are to be excluded from the lightweight and are instead to be included in the loading conditions as variable loads.

3.2.17 *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 floater.

3.2.18 *Mathieu instability:* Mathieu instability (MI) is a 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 or equal to half the natural period of pitch, causing the pitch motion to increase significantly.

3.2.19 *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 mean wind direction and rotor axis.

3.2.20 *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 TLPs are not referred to as mooring lines.

3.2.21 *Operational ballast water level:* Ballast water level in floater in the permanent operational condition on site. 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.

3.2.22 *Operational draught:* Draught of floater in the permanent operational condition on site. The draught that is intended to be used for inspection and repair might be a draught outside the range of operational draughts.

3.2.23 *P-delta effect:* The P-Δ or P-delta effect refers to the 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. The P-delta effect is a destabilizing moment equal to the force of gravity multiplied by the horizontal displacement a structure undergoes as a result of a lateral displacement.

3.2.24 *Power cable components:* All components that constitute the cable cross-section, such as conductors, insulation, tapes, armour wires, and sheath.

3.2.25 *Power cable system*: Complete power cable with terminations and permanent components, such as connector at floater interface, bend stiffener, buoyancy modules, and bend restrictors.

3.2.26 *Redundancy:* The ability of a component or system to maintain or restore its function when a failure of a member or connection has occurred. Redundancy may be achieved for instance by strengthening or introducing alternative load paths. 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 have redundancy.

3.2.27 *Return period:* The 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.

3.2.28 *Serviceability Limit States (SLS):* Disruption of normal operations due to deterioration or loss of routine functionality. 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.

3.2.29 *Set-down:* Set-down denotes the kinematic coupling between the horizontal surge/sway motions and the vertical heave motions.

3.2.30 *Shallow water:* Shallow waters are characterized by wavelengths greater than 10 times the water depth.

3.2.31 *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.

3.2.32 *Tendon:* Structural component used as part of the station keeping system for a TLP; also referred to as tether.

3.2.33 *Tension Leg Platform (TLP):* Vertically moored floating structure whose station keeping system consists of tethers or tendons anchored at the seabed.

3.2.34 *Ultimate Limit States (ULS):* Failure or collapse of all or part of structure due to loss of structural stiffness or exceedance of load-carrying capacity. Overturning, capsizing, yielding and buckling are typical examples of the ULS.

3.2.35 *Wave frequency motion:* Motion induced by first-order wave loads in the frequency range of the incoming waves.

3.2.36 *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.

3.2.37 *Wind heeling moment:* Product of wind force and wind force arm which produces the heel in question.

3.2.38 *Wind turbine:* System which converts kinetic energy in the wind into electrical energy. In this standard the term is used to designate the rotor–nacelle assembly.

3.2.39 *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.

4 Acronyms, Abbreviations and Symbols

4.1 Acronyms and abbreviations

4.1.1 Acronyms and abbreviations as given in Table 1-4 are used in this offshore standard.

Table 1-4 Acronyms and abbreviations

Short form	*In full*
3-T	Tension, time and temperature
C	Compliant
COG	Centre of Gravity
COV	Coefficient of Variation
DAE	Differential Algebraic Equation
DDF	Deep Draught Floater
DFF	Design fatigue factor
DOF	Degrees of Freedom
EEMUA	Engineering Equipment and Materials Users' Association
EOG	Extreme Operating Gust
FEM	Finite element method
FL	Floater
FMECA	Failure modes, effects and criticality analysis
FORM	First Order Reliability Method
FSD	Functional system design
GDW	Generalized Dynamic Wake
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
MWL	Mean Water Level
NA	Not Applicable
NDT	Non-destructive testing
OS	DNV Offshore Standard
PTI	Post-Tensioning Institute
QTF	Quadratic Transfer Function
R	Restrained
RMR	Rock Mass Rating
ROV	Remotely operated vehicle
RQD	Rock Quality Designation
SKS	Station Keeping System
SMTS	Specified minimum tensile strength
SMYS	Specified minimum yield strength
SWL	Still Water Level
TLP	Tension Leg Platform
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

4.2 Symbols

4.2.1 Latin characters

a	Coefficient
a_v	Vertical acceleration
b	Coefficient
c	Wave celerity
e	Void ratio
f	Frequency
g, g_0	Acceleration of gravity
h_o	Vertical distance
k	Wave number
n	Number of samples
n_C	Number of applied cycles
p	Pressure
p_d	Design pressure
p_e	Dynamic (environmental) sea pressure
p_s	Static sea pressure
p_{dyn}	Dynamic pressure
s_u	Undrained shear strength of clay
t	Time
u	Wind speed
z	Height, vertical distance
z_b	Vertical distance
A_{rotor}	Swept rotor area
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
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_S	Significant wave height
N	Number of sea states
N_C	Number of cycles to failure
Q	Variable functional load
R	Rotor radius
R	Resistance
R_c	Characteristic resistance
R_d	Design resistance
R_k	Characteristic resistance
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
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}	Reference wind speed for gust

4.2.2 Greek characters

α	Scale parameter
β	Shape parameter
γ'	Submerged unit weight of soil
γ_c	Consequence factor
γ_f	Load factor
γ_m	Material factor
φ	Friction angle
η	Utilization factor
θ	Angle
λ	Wavelength
μ	Mean value

ν	Poisson's ratio
ρ	Density of seawater, density of air
σ	Standard deviation
σ_d	Design material strength
σ_k	Characteristic material strength
σ_U	Standard deviation of wind speed
ω	Angular wave frequency
$\Delta\sigma$	Stress range

5 Typical Floaters and Boundary Conditions

5.1 General

5.1.1 Support structures for floating wind turbines may either be compliant, or restrained for some of the global modes of motions; surge, sway, heave, roll, pitch and yaw. For easy reference, Table 1-5 shows the different floater types with basis in floating offshore structures. This overview may not cover all possible solutions; however, the main types are believed to be captured. Restrained modes will not imply a total fixation, but displacements in the order of centimetres will be derived (e.g. an elastic stretch of a TLP tendon) compared to displacement in the order of metres for a compliant mode.

Table 1-5 Typical floaters and boundary conditions

Type	*Surge*	*Sway*	*Heave*	*Roll*	*Pitch*	*Yaw*
Deep Draught Floaters (DDF)[1)]	C	C	C	C	C	C
Semi submersibles	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
Truss structures	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 uses buoyancy as a vital part of the loadbearing system

The six global modes of motion referenced in Table 1-5 are illustrated in Figure 1-1.

In Table 1-5, C denotes compliant and R denotes restrained.

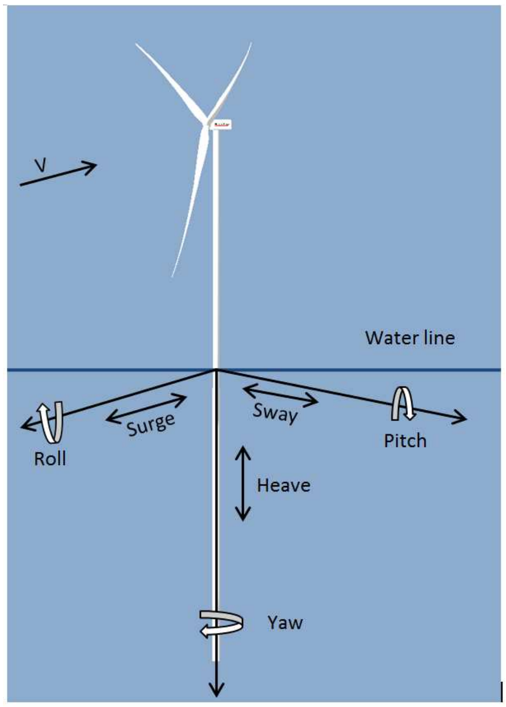

Figure 1-1
DOFs of a floating wind turbine

Surge	Translation along the longitudinal axis (main wind direction)
Sway	Translation along the lateral axis (transversal to the main wind direction)
Heave	Translation along the vertical axis
Roll	Rotation about the longitudinal axis
Pitch	Rotation about the lateral axis
Yaw	Rotation about the vertical axis

SECTION 2 SAFETY PHILOSOPHY AND DESIGN PRINCIPLES

1 Introduction

1.1 Objective

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

1.2 Application

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

2 Safety Philosophy

2.1 Safety class methodology

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

2.1.2 Three safety classes are defined. Low safety class is used for structures, whose failures imply low risk of human injury, minor environmental consequences, minor economic consequences and negligible risk to human life. Normal safety class is used for structures, whose failures imply some risk for human injury, some environmental pollution or significant economic consequences. High safety class is used for structures, whose failures imply large possibilities for human injuries or fatalities, for significant environmental pollution or major societal losses, or very large economic consequences. For floating wind turbine structures, which are unmanned during severe environmental loading conditions, the consequences of failure are mainly of an economic nature.

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

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 a safety class which is at least one safety class higher than the one specified in [2.1.3]. This implies that station keeping systems without redundancy shall be designed to high safety class.

Guidance note:

The issue of redundancy of the station keeping system is dealt with in Sec.8 [1.1].

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2.1.5 Design of secondary structures such as ladders, fenders and boat landings can be carried out to low safety class.

2.1.6 The different safety 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 safety class is applicable for a particular wind farm or structure in question.

2.1.7 The provisions given in [2.1.1] to [2.1.5] are based on the assumption that the floating wind turbine unit is unmanned during severe environmental loading conditions. It shall be assessed whether this assumption is fulfilled. When it is assessed that there is a risk that the floating wind turbine unit may be manned during severe environmental conditions, all components of the floating wind turbine unit and its station keeping system shall be designed to high safety class. This requirement can be waived if a risk mitigation to allow for normal unmanned operation can be implemented.

Guidance note:

Although the intention may be to keep the floating wind turbine unit unmanned during severe environmental conditions, manned units during such severe environmental conditions may be a possibility for wind farms located far away from any shore for which there may be a risk that maintenance personnel may be stranded on units in the wind farm in bad weather.

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2.2 Target safety

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^{-3} in low safety class, of 10^{-4} in normal safety class, and of 10^{-5} in high safety class. These target safety levels are aimed at for structures, whose failures are ductile, and which 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.

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2.3 Robustness

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

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.

Guidance note:

It takes time for technology to mature, maybe up to ten years, for example tested by experience.

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3 Design Principles and Design Conditions

3.1 Methods for structural design

3.1.1 The following design principles and design methods for limit state design of floating wind turbine structures are applied in this standard:

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

3.1.2 General design considerations regardless of design method are also given in [3.3.1].

3.1.3 This standard is based on the partial safety factor method, which is based on separate assessment of the load effect in the structure due to each applied load process. The standard allows for design by direct simulation of the combined load effect of simultaneously applied load processes, which is useful in cases where it is not feasible to carry out separate assessments of the different individual process-specific load effects.

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

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

3.2 Aim of the design

3.2.1 Structures and structural elements shall be designed to:

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

3.3 Design conditions

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.

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

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

4 Limit States

4.1 General

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

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 nonlinear environmental conditions

Serviceability limit states (SLS) corresponding to tolerance criteria applicable to intended use.

4.1.3 Examples of limit states within each category:

Ultimate limit states (ULS)

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

Fatigue limit states (FLS)

— cumulative damage due to repeated loads.

Accidental limit states (ALS)

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

Serviceability limit states (SLS)

— deflections that may alter the effect of the acting forces
— deformations that may change the distribution of loads between supported rigid objects and the supporting structure
— excessive vibrations producing discomfort or affecting non-structural components
— motion that exceed the limitation of equipment
— temperature-induced deformations.

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.

5 Design by the Partial Safety Factor Method

5.1 General

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.

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.

5.2 The partial safety factor format

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.

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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.

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

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

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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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 according to [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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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 [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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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.

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.

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.

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.

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.

5.3 Characteristic load effect

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

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

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.

5.4 Characteristic resistance

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

5.5 Load and resistance factors

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

6 Design Assisted by Testing

6.1 General

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

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.

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

6.2 Model tests

6.2.1 Model tests shall be carried out to validate software used in design, to check effects which are known not to be adequately covered by the software, and to check the structure if unforeseen phenomena should 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.

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

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.

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. When full-scale tests are not carried out as part of the design procedure, it is recommended to carry out the design to a safety class which is one safety class higher than the one required.

7 Probability-Based Design

7.1 General

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 [2.2.1] for the required safety class.

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.

7.1.3 For probability-based design, reference is made to specifications and requirements given in DNV-OS-J101.

SECTION 3 ENVIRONMENTAL CONDITIONS

1 Introduction

1.1 Definition

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.

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, soil conditions, seismicity, biology, and various human activities.

Guidance note:

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

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

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1.1.3 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.

1.1.4 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 [6].

1.1.5 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.

1.1.6 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.

1.1.7 For fatigue analysis and fatigue design, the concurrent combination of wind and waves is essential. The specification of an environmental class shall therefore include specification of the joint distribution of 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
- misalignment between wind and wave directions,

e.g. in terms of a scatter diagram for these quantities. Current-induced VIM/VIV may increase the wind and wave loads and should therefore be considered in this context.

1.1.8 The requirements for environmental conditions given in DNV-OS-J101 apply to floating wind turbine structures with the exceptions, deviations and additions specified in this section. Useful guidance regarding environmental conditions is given in DNV-RP-C205.

2 Wind, Waves and Current

2.1 General

2.1.1 This subsection gives requirements which come in addition to those given in DNV-OS-J101 regarding environmental conditions and which shall be fulfilled when floating structures are to be used to support offshore wind turbines.

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.

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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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 [2.1.3]. Adequate models for power spectral densities for waves and for wind are given in DNV-OS-J101 and in DNV-RP-C205.

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 DNV-OS-J101.

2.2 Wind

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 sec, 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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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.

2.2.3 The turbulence intensity is defined as the ratio σ_U/U_{10}.

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.

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, reference is made to DNV-OS-J101 and DNV-RP-C205, which also provide recommendations for the integral length scale that constitutes an important property of any power spectral density model.

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.

2.2.7 For some structures, 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 DNV-RP-C205 and IEC 61400-1.

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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2.2.9 A number of design load cases proposed in DNV-OS-J101 involve an Extreme Operating Gust (EOG) which is one of a number of defined reference wind conditions used in DNV-OS-J101 for definition of load cases. The EOG event presently specified in DNV-OS-J101 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.

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 DNV-OS-J101, Sec.4. The durations of the gusts considered shall be given relative to the critical natural periods, which typically are expected to be in the range from 10 to 100 sec, i.e. the durations of the gusts shall be so selected that they give rise to resonance of the floating wind turbine and its station keeping system.

Guidance note:

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

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

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 the reference wind speed of the gust event should be suitably chosen with a view to the expected dynamics for the floating unit in question 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.

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

2.3 Waves

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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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.

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.

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.

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.

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 DNV-OS-J101 and DNV-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.

2.3.7 The JONSWAP wave spectrum recommended in DNV-OS-J101 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 DNV-RP-C205.

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.

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 – can 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 nonconservative in deep waters.

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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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2.4 Current

2.4.1 For modelling of current, reference is made to DNV-OS-J101. For vortex-induced vibrations and vortex-induced motions, which can be an issue in deep waters, reference is made to DNV-RP-C205.

3 Water Depth and Water Level

3.1 Water depth

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

3.2 Water level

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 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 defined as the average of HAT and LAT.

Guidance note:

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

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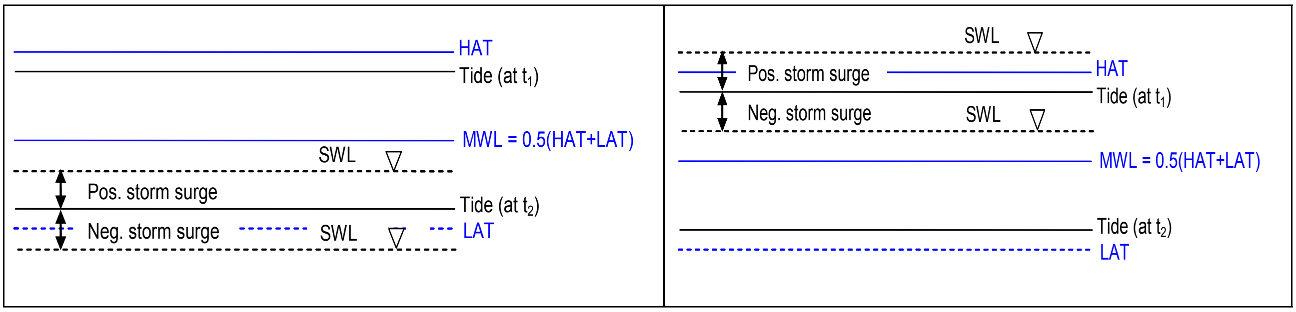

Figure 3-1
Definition of water levels when low still water level is governing (left) and when high still water level is governing (right)

3.2.2 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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3.2.3 Set-down effects associated with tendons may require special focus on water level with consideration of effects of tides, storm surges and tsunamis.

4 Seismicity

4.1 General

4.1.1 Reference is made to DNV-OS-J101, Sec.3, Item H100. For details of seismic design criteria, reference is made to ISO 19901-2.

4.1.2 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.

4.1.3 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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5 Soil Conditions

5.1 General

5.1.1 For design of floaters whose structural design depends on the soil conditions, a range of soil conditions shall be defined as part of the definition of an environmental class for the design of the floater.

5.1.2 For design of station keeping systems and their components, such as anchors and mooring lines, a range of soil 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.

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, soil 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.

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 – 30°	8.5 – 11.0	0.35	0.7 – 0.9
Medium	30 – 36°	9.0 – 12.5	0.35	0.5 – 0.8
Dense	36 – 41°	10.0 – 13.5	0.35	0.4 – 0.6

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 s_u (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

5.1.6 Site-specific soil investigations form the basis for establishing soil conditions for a particular wind farm project. For soil investigations, requirements and recommendations given in DNV-OS-J101 apply.

6 Regional Environmental Data for Definition of Environmental Classes

6.1 General

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.

6.1.2 For definition of environmental classes, key environmental wave parameters for various regions in the world are given in App.B Table B-1. The significant wave heights tabulated in Table B-1 refer to 3-hour stationary sea states. For stationary sea states with duration other than three hours, the significant wave height data in Table B-1 must be suitably converted to properly refer to the actual sea state duration, see [6.1.3]. The data in Table B-1 refer to wave heights with specific return periods. Wave heights with other return periods can be found by interpolation. Wave height data for regions not covered by Table B-1 can be found in DNV-RP-C205, Appendix C, and include Weibull scale and shape parameters for regional long-term distributions of significant wave height.

6.1.3 The significant wave heights with specified return periods given in Table B-1 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 1000 years, which can be extracted from Table B-1.

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 according to the following expression

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

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. When data for H_S from Table B-1 are to be converted, T_S=3 hours and N=2922.

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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 App.B Table B-2.

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. Reference is made to [2.3.6].

6.1.6 For definition of environmental classes, key environmental wind parameters for various regions in the world are given in App.B Table B-3. The mean wind speeds tabulated in Table B-3 are 10-minute mean wind speeds, i.e. they refer to a 10-minute averaging period. Formulas for conversion of wind data from the 10-minute averaging period in Table B-3 to other averaging periods, retaining the return periods, are given in DNV-OS-J101.

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. Reference is made to [2.2.5].

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.

6.1.9 The regional environmental data given in the tables in App.B and referenced in [6.1.2] through [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.

7 Other Site Conditions

7.1 Salinity

7.1.1 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.

7.2 Temperature

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.

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

7.3 Marine growth

7.3.1 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 DNV-OS-J101. 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.

SECTION 4 LOADS AND LOAD EFFECTS

1 Introduction

1.1 General

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.

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. For further details, reference is made to DNV-RP-C205 and DNV-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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1.2 Extreme loads

1.2.1 Operational conditions during power production will often produce the governing extreme loads which will then be dominated by the thrust force formed by the rotor-filtered 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 moments. The highest mooring line loads are often governed by wave loads.

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.

1.3 Fatigue loads

1.3.1 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.

1.4 Transportation loads

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

1.5 Wind turbine loads

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.

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.

1.6 Wake-induced loads

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

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.

2 Response Characteristics

2.1 General

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.

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.

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.

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. 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.

2.2 Deep-draught floaters (DDFs)

2.2.1 Although DDFs such as 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 Sec.7 [1.3].

2.3 Semisubmersibles

2.3.1 Semisubmersibles are usually column-stabilized units consisting of large-diameter support columns attached to submerged pontoons. The pontoons may be of different designs such as ring pontoons, twin pontoons and multi-footing pontoons. Semisubmersibles 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 semisubmersible 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 semisubmersible concept.

2.4 Tension-leg platforms (TLPs)

2.4.1 A 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.

2.4.2 A TLP generally experiences WF motions in the horizontal plane that are of the same magnitude as those of a semisubmersible 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.

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

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 DNV-OS-C105.

2.5 Monohull structures

2.5.1 Due to 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.

3 Basis for Definition of Characteristic Loads

3.1 Load categorization

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.

3.1.2 The following load categories are defined:

— permanent loads
— variable functional loads
— environmental loads
— abnormal wind turbine loads (loads associated with fault situations for the wind turbine)
— deformation loads
— accidental loads.

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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3.2 Definition of characteristic loads

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 [3.2.2] and [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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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, the weather forecast and the consequences of failure. For design conditions during transport and installation, reference is made to DNV-OS-H101 and DNV-OS-H102.

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		
Environmental (E); weather restricted	Specified value	Expected load history	Not applicable	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		

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

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	Not applicable	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				

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	

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 Permanent Loads (G)

4.1 Definition

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.

4.1.2 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.

4.1.3 Permanent pressures are dealt with together with other loads on the hull structure in [9].

4.2 Floater-specific issues

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.

4.2.2 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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4.2.3 Permanent solid and liquid ballast used for floating stability purposes is categorized as a permanent load.

4.2.4 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.

5 Variable Functional Loads (Q)

5.1 Definition

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
— ship impacts from service vessels due to normal operation
— weight of variable ballast and pressures due to variable ballast
— crane operational loads.

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.

5.1.3 Ship impacts are dealt with in [5.2]. Pressures due to variable ballast are dealt with together with other loads on the hull structure in [9]. For details about other variable loads mentioned in [5.1.1], reference is made to DNV-OS-J101.

5.2 Ship impacts and collisions

5.2.1 Ship impact loads due to normal operations are defined as variable functional loads. Requirements regarding ship impacts as specified in DNV-OS-J101 apply to floating wind turbine structures unless otherwise specified herein.

5.2.2 Ship impact loads as described in DNV-OS-J101 may need to be supplemented as necessary for floating support structures, for example if larger ships have to be considered in predicting loads from ship impacts due to normal operation. The design against ship 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.

5.2.3 Ship 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 ship 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 ship collision are governed by the ratio between the stiffness of the ship and the stiffness of the floater and by the ratio between the mass of the ship and the mass of the floater. The lateral compliance of the floating structure is no guarantee that the result of a ship 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 ship collision more severe than for a bottom-fixed structure.

5.2.4 A model for ship impact analysis is given in DNV-RP-C104, Sec.8.

6 Environmental Loads (E)

6.1 Definition

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.

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.

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

6.1.4 Practical information regarding environmental conditions and environmental loads are given in DNV-RP-C205.

6.2 Floater-specific issues

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 standard default 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.

The need for simulation times as long as 3 to 6 hours is first of all expected for analysis in the operating condition. For analysis in the survival condition, where the wind turbine is not operating, shorter simulation times are expected to suffice.

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 Sec.3 [6.1.3] and conversion formulas for mean wind speed in

DNV-OS-J101, Sec.3. 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.

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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.

6.3 Design load cases

6.3.1 DNV-OS-J101, Sec. 4, Table E1, provides a number of proposed load cases by combining various environmental conditions. This table 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.

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

6.4 Ice loads

6.4.1 Proposed load cases for ice loading, given in DNV-OS-J101, Sec.4 E500, 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.

6.5 Fatigue loads

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 can be necessary.

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, such as sensor errors in the turbine control system, 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.

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

7 Accidental Loads (A)

7.1 Definition

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 1000-year loads, i.e. loads with an annual probability of exceedance of 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.

7.1.2 Accidental loads from unintended collisions with drifting service vessels shall be considered and shall adhere to [5.2.3] and shall meet the requirements specified in DNV-OS-J101, Sec.4, D300.

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.

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 reckoned as accidental loads.

8 Deformation Loads (D)

8.1 Definition

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 DNV-OS-E303.

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8.1.2 Requirements for deformation loads are given in DNV-OS-J101.

9 Pressure Loads on Hull

9.1 General

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.

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 [9.2] and [9.3], respectively. Typical combinations for local tank pressures and sea pressures are addressed in [9.4].

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 DNV-OS-C401, Ch.2, Sec.4, can be applied.

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.

9.2 Tank pressures

9.2.1 Tanks shall be designed for the maximum filling height. Three alternative tank filling types are defined as specified in [9.2.2], [9.2.3] and [9.2.4], respectively. Requirements for the design tank pressures for these three alternatives are given in [9.2.5]. Additional requirements for a fourth alternative, referring to tanks that can be emptied by air pressure, are specified in [9.2.6].

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.

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.
- Criteria applicable for the tank filling arrangements are given in DNV-OS-D101 Ch.2 Sec.3 C300. 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 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 DNV-OS-D101 Ch.2 Sec.3 C300, for free flooding ballast systems.
- The dynamic pressure head due to the operation of the pumps, p_{dyn}, may be neglected.

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 due to the operation of the pumps, p_{dyn}, should be considered.

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 = 9.81m/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.

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 [9.2.6] need not be combined with extreme environmental loads. Normally only static global response need to be considered.

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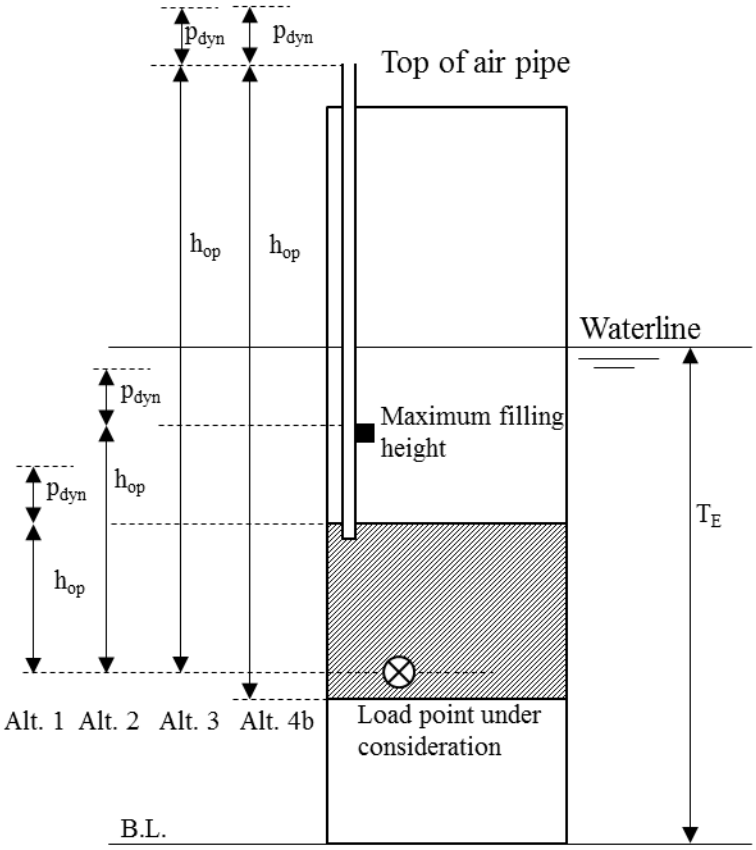

Figure 4-1
Parameters for internal tank pressures

9.3 Sea pressures for the ULS

9.3.1 The design sea pressure acting on floating wind turbine units in operating conditions in deep waters can 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 (B.L.) to the assigned load waterline. Set-down effects should be considered.
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)

For shallow water depths the design sea pressure can be calculated according to DNV-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.

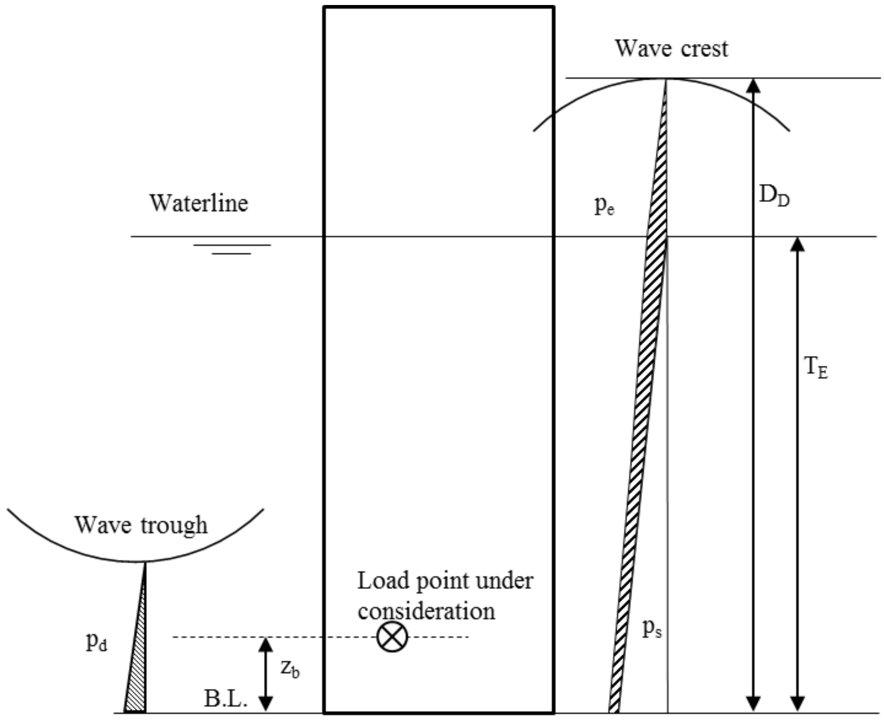

Figure 4-2
Parameters for sea pressures

9.4 Combination of tank pressures and sea pressures

9.4.1 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).

9.5 Superimposition of responses

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.

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.

9.5.3 Further information regarding superimposition of loads from local and global models can be found in DNV-RP-C103.

9.6 Wave slamming

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.

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 in the splash zone is a local load effect which usually does not compromise the global structural capacity.

9.6.3 For further information regarding wave slamming and its representation, reference is made to DNV-RP-C205.

SECTION 5 LOAD FACTORS AND MATERIAL FACTORS

1 Load Factors

1.1 Load factors for the ULS and the ALS

1.1.1 Table 5-1 provides four sets of load factors to be used when characteristic loads or load effects from different load categories are combined to form the design load or the design load effect for use in design. For analysis of the ULS, the sets denoted (a) and (b) shall be used when the characteristic environmental load or load effect is established as the 98% quantile in the distribution of the annual maximum load or load effect. For analyses of the ULS for abnormal wind load cases as defined in DNV-OS-J101, the set denoted (c) shall be used. For analysis of the ALS, the load factor set denoted (d) 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 safety 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 set (b) in Table 5-1, see Sec.8.

Table 5-1 Load factors γ_f for the ULS and the ALS

Load factor set	*Limit state*	*Load categories*					
		G	*Q*	*E*			*D*
				Safety class			
				Low	*Normal*	*High*	
(a)	ULS	1.25	1.25	0.7 (*)			1.0
(b)	ULS	1.0	1.0	1.20	1.35	1.55	1.0
(c)	ULS for abnormal wind load cases	1.0	1.0	1.1			1.0
(d)	ALS	1.0	1.0	0.9	1.0	1.15	1.0

Load categories are:

G = permanent load

Q = variable functional load, normally relevant only for design against ship impacts and for local design of platforms

E = environmental load

D = deformation load.

For description of load categories, see Sec.4.

(*) When environmental loads are to be combined with functional loads from ship impacts, the environmental load factor shall be increased from 0.7 to 1.0 to reflect that ship impacts are correlated with the wave conditions.

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 ship 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 ship impact load from a service vessel with an environmental load according to load factor set (a), the characteristic environmental load will have to be taken as the 98% quantile in the distribution of the annual maximum environmental load conditioned on the environment being below the specified threshold for operation of the service vessels. This is a load which will be smaller than the characteristic environmental load for the situation that no weather restrictions are in place. The distribution of the annual maximum environmental load conditioned on the environment being below the specified threshold for operation of the service vessels 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.

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1.1.2 The characteristic environmental load effect (E), which forms part of the load combinations of Table 5-1, is to be taken as the characteristic combined load effect, determined according to Sec.4, and representing the load effect that results from two or more concurrently acting load processes.

1.2 Load factor for the FLS

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.

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

1.3 Load factor for the SLS

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

2 Material Factors

2.1 Material factors for the ULS

2.1.1 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 foundations which qualify for design to normal safety class.

2.2 Material factors for the FLS

2.2.1 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.

2.3 Material factors for the ALS and the SLS

2.3.1 The material factor γ_m for the ALS and the SLS shall be taken as 1.0.

SECTION 6 MATERIALS

1 Introduction

1.1 General

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 DNV-OS-J101.

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.

1.1.3 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).

1.1.4 Material designations are defined in DNV-OS-J101.

2 Selection of Metallic Materials

2.1 General

2.1.1 For selection of structural steel for design and construction of floating support structures, reference is made to the materials section of DNV-OS-J101.

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 Sec.6 [6].

2.2 Structural steel and aluminium

2.2.1 Special material requirements to support structural integrity are given in [2.2.2] to [2.2.4].

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 DNV-OS-J101.

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

2.2.4 Aluminium shall be of seawater resistant type. Aluminium alloys shall comply with the requirements set forth in DNV-OS-B101.

2.3 Bolting materials

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.

Guidance note:
Bolts in accordance with ASTM A 320 Grade L7 are acceptable within given limitations.

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3 Selection of Concrete Materials

3.1 General

3.1.1 For selection of concrete materials for design and construction of floating support structures, reference is made to the materials section of DNV-OS-J101. Useful information for selection of concrete materials can also be found in DNV-OS-C502.

4 Selection of Grout Materials

4.1 General

4.1.1 For selection of grout materials for design and construction of grouted connections in floating support structures, reference is made to the materials section of DNV-OS-J101.

5 Selection of Composite Materials

5.1 General

5.1.1 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 DNV-OS-C501.

6 Selection of Materials for Chains, Wires and Tethers

6.1 General

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.

6.1.2 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.

6.1.3 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.

6.1.4 Requirements for system analysis are found in DNV-OS-E301.

6.2 Chains

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 five grades: R3, R3S, R4, R4S and R5. Specific requirements for each grade are given in DNV-OS-E302, Ch.2, Sec.1.

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

6.3 Steel wires

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 6-1.

Table 6-1 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

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

6.4 Fibre ropes

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.

Guidance note:

The change-in-length performance is the length and dynamic stiffness of the fibre rope as function of loading sequence and time.

The loadbearing capability of synthetic-yarn materials is referred to as 3-T (triple-T) since it depends on the combination of the critical parameters ‘tension’, ‘time’ and ‘temperature’.

The Nominal Loadbearing Linear Density is the aggregate linear density of the loadbearing yarns as measured at the yarn manufacturer. There is no testing of rope required to determine nominal loadbearing linear density.

The basic measurement unit for linear density, i.e. weight per unit length of textiles is tex = $10^{-3} \times$ g/m; and dtex = $10^{-4} \times$ g/m, ktex = g/m, and Mtex = kg/m.

The length of the fibre rope as it has been produced is denoted as the Original length. It is measured at reference tension, either as the gauge length during testing or as the Original rope length during the process of manufacturing for delivery.

Change-in-length effects include, but are not limited to, creep.

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6.4.2 Requirements for offshore fibre ropes and tethers are given in DNV-OS-E303. Recommendations will be provided in *DNV-RP-E304* for condition management and in *DNV-RP-E305* for design, testing and analysis.

7 Selection of Materials for Electrical Cables

7.1 General

7.1.1 Requirements for material selection for electrical cables are given in ISO 13628-5 and shall apply.

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

8 Selection of Materials for Solid Ballast

8.1 General

8.1.1 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.

SECTION 7 STRUCTURAL DESIGN

1 Introduction

1.1 General

1.1.1 The requirements for structural design given in DNV-OS-J101 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 DNV-OS-J101 apply unless otherwise specified in this section.

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 DNV-RP-C205 and DNV-RP-F205.

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.

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

1.2 Interface with wind turbine

1.2.1 The rotating turbine will influence the global motions of the floating wind turbine in the operating mode, mainly in roll and pitch. 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 are 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.

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.

Guidance note:

An optimum floater design is necessary to achieve a cost-effective solution for offshore floating wind turbines.

One way of achieving an optimum floater design is to utilize available response analysis programs for mooring system forces and vessel motions in combination with a method for solution of nonlinear optimization problems with arbitrary constraints. Examples of constraints to consider include vessel motion, tower inclination, tower top acceleration, draught, mooring line tension, minimum horizontal pretension, and maximum horizontal offset.

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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.

1.3 Design against undesirable effects

1.3.1 It shall be ensured that effects like Mathieu Instability (MI) and Vortex Induced Motions (VIM) are unlikely to occur or can be controllable. 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:

DNV-OS-C106 and DNV-RP-F205 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.

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1.4 Installation-friendly design

1.4.1 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 structural design is carried out with a view to allow for use of the same weather criterion for installation of support structures and mooring lines as for wind turbines and cables. The risk for delays associated with use of different weather criteria for installation of different components is the key issue here. The same weather criterion for installation of different structural components can be achieved by planning and suitable design.

However, if some operations can be performed in higher waves than other operations, the implication may be a larger flexibility of the installation activities, provided none of these activities are subject to too stringent restrictions.

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1.5 Special provisions for global analysis

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.

1.5.2 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.

1.6 Special provisions for semisubmersibles

1.6.1 General considerations with respect to methods of analysis and capacity checks of semisubmersibles are given in DNV-OS-C103 and DNV-RP-C103.

1.6.2 The maximum responses in a semisubmersible 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 7-1.

1.6.3 For preliminary design and for design in the survival condition, 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 semisubmersible)
— 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.

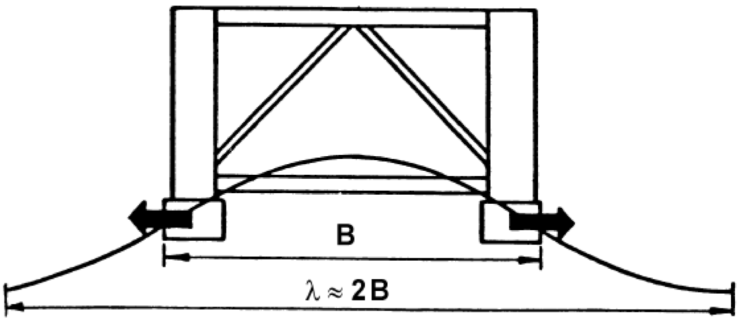

Figure 7-1
Split force between pontoons

1.6.4 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. DNV-OS-J101 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.

1.6.5 Analysis undertaken to check air gap should be calibrated against relevant model test results when available, see DNV-OS-C103 Sec.4 D102.

1.6.6 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.

1.6.7 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 DNV-OS-E301 Ch.2 Sec.4. 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 DNV-OS-C103 Sec.7 B100. 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 Sec.8 [2].

1.6.8 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 [1.6.7])
The material factor γ_m is 1.0 in this case.

b) Operational loads from all mooring lines:

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.

1.7 Special provisions for TLPs

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

1.7.2 The maximum responses in a TLP are often not governed by the maximum wave height and associated wave period. Reference is made to [1.6].

1.7.3 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.

1.7.4 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, due to thrust in operating condition, due to mean wind in survival condition, or due to drift load)
- set-down (due to current, due to thrust in operating condition, due to mean wind in survival condition, or due to drift load)
- WF tension (wave frequency component)
- LF tension (wind gust and slowly varying drift)
- ringing (HF response)
- seismic risk
- hull VIM influence on tendon responses
- tendon VIV induced loads.

1.7.5 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).

1.7.6 Composite tendons shall be designed in accordance with DNV-OS-C501 with additional provisions as given in this standard.

1.7.7 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 safety class as specified in Sec.2 [2.1.5].

1.7.8 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.

1.7.9 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 Sec.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.

1.7.10 Tendons and tendon components constructed from steel shall be designed against the FLS with a design fatigue factor DFF for steel equal to or greater than given in Table 7-2 for case (d) and case (e) as applicable.

1.7.11 For tendons constructed by fibre ropes, design against the FLS shall be carried out according to requirements given in DNV-OS-E301, DNV-OS-E303 and *DNV-RP-E305*.

1.7.12 Guidelines for certification of tendons are given in DNV-OS-C105 Appendix A.

1.8 Special provisions for spars

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

1.8.2 When tendons are used for station keeping of a spar type unit, the provisions specified for tendons in [1.7] apply.

1.8.3 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.

1.8.4 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.

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1.8.5 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.

1.8.6 For deep draught floaters, 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.

1.8.7 Requirements for the position mooring system are given in Sec.8 and in [1.6].

1.9 Provisions for concrete structures

1.9.1 Floating concrete support structures for floating wind turbines shall be designed according to DNV-OS-J101, which makes reference to DNV-OS-C502. Requirements for partial safety factors shall be taken according to specifications given in DNV-OS-J101.

1.9.2 Concrete components of floating support structures for wind turbines shall be designed according to DNV-OS-J101, which makes reference to DNV-OS-C502. Requirements for partial safety factors shall be taken according to specifications given in DNV-OS-J101.

1.10 Special provisions for towers

1.10.1 Even when the mass and stiffness distributions over the height of the tower are kept the same as for the tower for an onshore wind turbine, the eigenfrequencies of the tower are altered when it is mounted on a floater. This is because the bottom boundary condition of the support structure, except in the case of a TLP, is altered from fixed to free. In order to keep the eigenfrequencies of the tower unchanged, equal to those of the tower supporting the onshore turbine, the tower should be made more flexible when it is to be supported by a floater like a spar or a semisubmersible. Reference is made to guidance note in Sec.1 [1.1.5].

1.10.2 The tower access system consists of secondary structures such as boat landings, ladders and platforms. Requirements for such secondary structures are given in DNV-OS-J101.

2 Ultimate Limit States (ULS) for Steel Structures

2.1 General

2.1.1 Provisions and requirements for design of various types of structures and structural components against ultimate limit states (ULS) are given in [2.2] through [2.6].

2.2 ULS – Tubular members, tubular joints and conical transitions

2.2.1 Tubular members, tubular joints and conical transitions shall be designed according to 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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2.3 ULS – Shell structures

2.3.1 Except for towers, shell structures shall be designed against buckling in accordance with DNV-RP-C202. Shell structures in towers shall be designed against buckling according to DNV-OS-J101.

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 can always be considered in these checks. Only in the case that a horizontal earth pressure from saturated solid ballast can be documented, such an earth pressure can be accounted for in the buckling checks.

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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2.4 ULS – Non-tubular beams, columns and frames

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.

2.4.2 Buckling checks may be performed according to EN 1993-1-1.

2.4.3 Capacity checks may be performed according to EN 1993-1-1.

2.4.4 The material factors according to Table 7-1 shall be used when EN 1993-1-1 is used for calculation of structural resistance.

Table 7-1 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 according to EN 1993-1-1.		

2.5 ULS – Special provisions for plating, stiffeners and girders

2.5.1 For minimum requirements for plate thickness, stiffener sectional modulus and girders, reference is made to DNV-OS-J101. These requirements will normally give minimum scantlings to plates and stiffened panels with respect to yield.

2.5.2 Buckling of stiffened plates including girders shall be checked according to DNV-RP-C201.

2.6 ULS – Special provisions for structural design of anchors

2.6.1 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 according to specifications given in Sec.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.

2.6.2 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 according to specifications of characteristic loads given in Sec.4 and requirements for load factors and load combinations given in Sec.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.

2.6.3 Caution must 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 Sec.9 [1.2.3] and [1.2.4].

2.7 ULS – Bolted connections

2.7.1 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.

2.7.2 Slip-resistant bolt connections shall be designed according to DNV-OS-J101.

2.7.3 End plate bolt connections shall be designed according to EN 1993-1-8.

3 Fatigue Limit States (FLS) for Steel Structures

3.1 General

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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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.

3.1.3 Complex connections of plated structures shall be documented according to the hot spot methodology described in DNV-RP-C203 Sec.4.

Guidance note:

Complex joints are details not explicitly covered by DNV-RP-C203, i.e. joints other than those covered by DNV-RP-C203, Appendices A to C.

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3.1.4 Characteristic S-N curves for use in design against fatigue failure are given in DNV-OS-J101.

Guidance note:

In general, the classification of structural details and their corresponding S-N curves in air, in seawater with adequate cathodic protection and in free corrosion conditions, can be taken from DNV-RP-C203 "Fatigue Strength Analyses of Offshore Steel Structures".

The S-N curves for the most frequently used structural details in steel support structures for offshore wind turbines are given in DNV-OS-J101, Sec.7, Table J1. Which of these S-N curves are to be used in design depend on location zone for the structural detail and on the corrosion protection of the structural surface and are given in DNV-OS-J101, Sec.7, Table J2.

Curves specified for material in air are valid for details, which are located above the splash zone. The "in air" curves may also be used for the internal structures of air-filled members below water.

The basis for the use of the S-N curves in Table J1 in DNV-OS-J101 Sec.7 is that a high fabrication quality of the details is present, i.e. welding and NDT shall be in accordance with Inspection Category I and Structural Category 'Special' according to DNV-OS-C401.

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3.1.5 Corrosion shall be taken into consideration where relevant. Both the corrosion diminution 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 relevant:

— The structure is protected against corrosion for the whole service life: no reduction in thickness and no change in S-N curve
— 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".
— 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.

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 DNV-OS-J101, Sec.11.

The additional costs of corrosion allowance for replaceable secondary structures should be balanced against the costs of replacement.

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 DNV-RP-C203. An alternative method for fracture mechanics analysis can be found in BS 7910.

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 DNV-RP-C203 and DNV Classification Notes No. 30.7.

It is recommended that fabrication tolerances in accordance with “Special” category in DNV-OS-C401 are used to minimize the stress concentration factor for butt welds.

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3.1.8 For fatigue analysis of both base material and welded structures, 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 DNV-OS-J101.

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

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

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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*

$$D_D = DFF \cdot D_C$$

3.1.11 Requirements for the design fatigue factor DFF are given in Table 7-2 and depend on the required safety class. The design fatigue factors in Table 7-2 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.

Guidance note:

Non-accessible and non-repairable structural details will have to be designed separately from accessible and repairable details. The accessible and repairable details will then have to be designed under the assumption of inspections every four or five years. 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.

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Table 7-2 Requirements for design fatigue factors, DFF

Structural element	*Safety class*		
	Low	*Normal*	*High*
(a) Internal structure, accessible and not welded directly to the submerged part.	1	2	3
(b) External structure, accessible for regular inspection and repair in dry and clean conditions.	1	2	3
(c) Internal structure, accessible and welded directly to the submerged part.	2	3	6
(d) External structure not accessible for inspection and repair in dry and clean conditions.[1)]	2	3	6
(e) Non-accessible areas, areas not planned to be accessible for inspection and repair during operation, and structures with permanent ballast.[2)]	3	6	10
1) Regular inspection, preferably by NDT.			
2) No planned inspection.			

3.1.12 The design criterion is

$$D_D \leq 1.0.$$

3.1.13 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 according to the conditions specified in DNV-OS-J101 and in DNV-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 DNV-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.

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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4 Accidental Limit States (ALS)

4.1 General

4.1.1 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 [4.1.2] to [4.1.7].

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

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

— check of resistance of the structure against design accidental loads
— check of post-accident resistance of the structure against environmental loads when the structural resistance has become reduced by structural damage caused by the design accidental loads such as the design fire or the design collision.

4.1.4 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.

4.1.5 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 accidents. 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.

4.1.6 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.

4.1.7 Typical accidental loads are listed in Sec.4.

4.2 Post-accidental integrity after unintended change in ballast distribution

4.2.1 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.

4.3 Post-accidental integrity of a shared anchor

4.3.1 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.

SECTION 8 STATION KEEPING

1 Introduction

1.1 General

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

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.

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 the safety class which is specified in Sec.2 [2.1.4]. This requirement refers to station keeping systems which have redundancy.

1.1.4 For station keeping systems without redundancy, all structural components in the station keeping system shall be designed to a higher safety class as specified in Sec.2 [2.1.5]. 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 relevant safety class.

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1.1.5 The requirement in [1.1.4] can 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 safety class specified in Sec.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 can 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.

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1.2 Compliant floaters – general

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 soil conditions on the actual site.

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 safety class according to [1.1.3] to [1.1.5].

Guidance note:

Safety factor requirements are dependent on safety class and safety class is dependent on consequence of failure which in turn is dependent on redundancy of system. In full analogy, DNV-OS-E301 (Position Mooring) 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.

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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.

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.

1.2.5 Requirements for design of mooring lines are given in [2]. For further details of principles for design of mooring lines, reference is made to DNV-OS-E301.

1.3 Restrained floaters – general

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.

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.

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.

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.

1.3.5 Requirements for design of tethers are given in [3] which refers to Sec.7. For further details of principles for design of metallic tethers, reference is made to DNV-OS-C105. For further details of principles for design of fibre ropes, reference is made to DNV-OS-E303.

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 due 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.

1.3.7 Redundancy considerations are an important part of tendon design and form part of the basis for selection of the appropriate safety class according to [1.1.3], [1.1.4] and [1.1.5].

Guidance note:

In analogy with the requirements in [1.1.3] to [1.1.5], for tendon systems without redundancy with respect to loss of a tether, the standard DNV-OS-C105 for TLP design recommends that load factors used for tether design be increased by 20% relative to those used for redundant systems.

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2 Mooring Lines

2.1 Introduction

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, reference is made to [3].

2.2 Ultimate loads

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.

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.

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 can 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 according to a methodology described in NORSOK N-006, Sec. 6.6.

2.2.4 The prerequisite for the procedure in [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.

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 DNV-OS-E301.

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

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

Limit state	*Load factor*	*Safety class*	
		Normal	*High*
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

2.3 Resistance

2.3.1 The characteristic capacity of a mooring line is defined in [2.3.5]. The premises for the definition are given in [2.3.2] to [2.3.4].

2.3.2 The characteristic capacity specified in [2.3.5] is defined on the following basis:

— The mooring line components shall be manufactured with a high standard of quality control, according to recognized standards, such as DNV-OS-E302, DNV-OS-E303 and DNV-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.

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 [2.3.5].

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.

2.3.5 The characteristic capacity of the body of the mooring line constructed from the component with the properties specified in [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.

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

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

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 DNV-OS-E302, DNV-OS-E303 and DNV-OS-E304.

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.

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.

2.3.10 For definition of the characteristic capacity of anchors for transfer of mooring line forces to the seabed soils, reference is made to Sec.9.

2.4 Design criterion

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 [2.3.8].

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 [2.3.8], and in which T_d is established under an assumption of damaged mooring system in terms of one broken mooring line.

2.5 Fatigue design

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* depend on the safety class and are given in Sec.7. 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.

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 DNV-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.

2.5.3 For simplicity, the characteristic cumulative fatigue damage may be calculated by means of the simple conservative "combined spectrum approach" outlined in DNV-OS-E301.

2.5.4 The design criterion is

$$D_D \le 1.0.$$

3 Tendons

3.1 Steel tendons

3.1.1 Requirements for design of steel tendons are given in [3.3] and in Sec.7. Load factor requirements for steel tendons are given in Sec.5 [1.1].

3.2 Fibre rope tendons

3.2.1 The design tension, T_d, in a fibre rope tendon shall be calculated according to specifications of characteristic loads given in Sec.4 and requirements for load factors and load combinations given in Sec.5 [1.1].

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 $\gamma_m = 1.5$ under an assumption of ductile material behaviour and not more than 10% coefficient of variation of the tensile capacity.

3.2.3 The design criterion in the ULS is

$$R_d > T_d$$

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 DNV-OS-E301 and DNV-OS-E303.

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 [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; tension, time, temperature) shall be demonstrated.

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 DNV-OS-E301 and in DNV-OS-E303. Further recommendations will be provided in *DNV-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 DNV-OS-E303.

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3.3 Tendon slack

3.3.1 Tendons shall be designed against slack in the ULS.

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, and $T_{dynamic,c}$ is the characteristic environmental dynamic force in the tendon. Requirements for the load factors are given in Sec.5 [1.1]. Usually the load factor set denoted ULS (b) is the set that governs the design against tendon slack.

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3.3.2 The requirement in [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.

3.3.3 When temporary tendon tension loss is permitted in the ULS according to [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

1 Introduction

1.1 General

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.

1.1.2 The requirements for foundation design given in DNV-OS-J101 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. In particular, the requirements for material factors specified in DNV-OS-J101 apply unless otherwise specified in this section.

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
— grouted 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 turbines, but not as shared anchor points for anchoring of multiple wind turbines.

1.1.4 Type-specific requirements for anchor design are given in [2] through [8].

1.1.5 The analysis of anchor resistance shall be carried out for the ULS and the ALS and 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.

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.

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.

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.

1.2 Anchor load

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 according to specifications given in Sec.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.

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 according to specifications of characteristic loads given in Sec.4 and requirements for load factors and load combinations given in Sec.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.

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.

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.

1.3 Anchor resistance

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.

1.3.2 When the characteristic value of a soil property or an anchor resistance is estimated from limited data, the estimate shall be a cautious estimate. This implies that the characteristic value shall be estimated with confidence. It is recommended to apply a confidence of at least 75%.

Guidance note:

Relevant statistical methods should be used for estimation of characteristic values of soil properties. For estimation of characteristic values of soil properties by means of statistical methods, reference is made to DNV-RP-C207.

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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 soil conditions, shape and size of anchors, and loading conditions.

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.

1.3.5 Requirements for the material factor are given for the individual anchor types in [2] through [8].

1.4 Design criterion

1.4.1 The design criterion is

$$T_d \le R_d$$

1.5 Effects of cyclic loading

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.

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.

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:

Cyclic degradation of soil properties is the issue here, as well as rate effects. 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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2 Pile Anchors

2.1 General

2.1.1 Pile anchors shall be designed in accordance with the relevant requirements given in DNV-OS-J101 Sec.10. Useful guidance is given in Classification Notes No. 30.4.

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

2.2 Material factors

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 safety class and regardless of the material factors specified in DNV-OS-J101.

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 [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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2.3 Anchors in chalk

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.

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 due 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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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.

3 Gravity Anchors

3.1 General

3.1.1 Gravity anchors shall be designed in accordance with the relevant requirements given in DNV-OS-J101 Sec.10.

3.1.2 The capacity against uplift of a gravity anchor shall not be taken higher than the submerged mass of the anchor. However, for anchors furnished with skirts penetrated into the seabed soils, the contribution from soil friction along the skirts may be included. In certain cases such anchors may be able to resist cyclic uplift loads by the development of temporary suction within their skirt compartments. In relying on such suction one shall make sure that there are no possibilities for leakage, e.g. through pipes or leaking valves or channels developed in the soil, which could prevent the development of the suction.

3.2 Material factors

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 safety class.

4 Suction Anchors in Clay

4.1 General

4.1.1 Suction anchors are vertical cylindrical anchors with open or (normally) 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 shear strength profile, and whether the anchor has an open or a closed top.

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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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.

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.

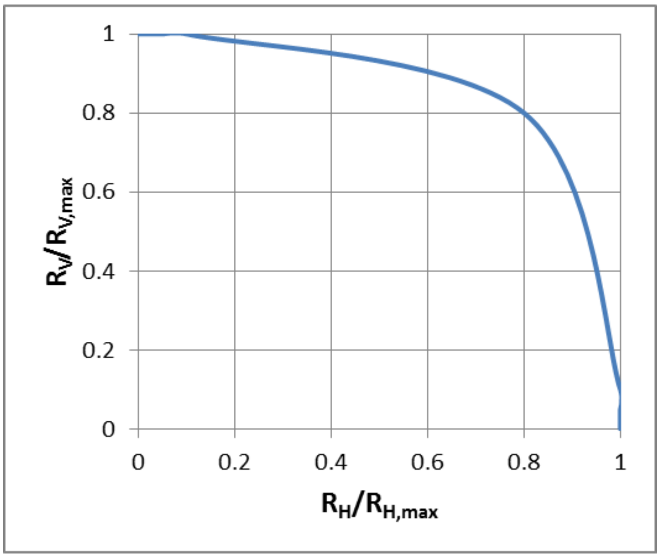

Figure 9-1
Schematic resistance diagram for suction anchor, example

4.1.4 Recommendations for geotechnical design and installation of suction anchors in clay are provided in the recommended practice DNV-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. For details, see DNV-RP-E303.

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.

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.

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. A confidence of minimum 75% is recommended for the estimation and due account shall be made of the quality of the soil data and the complexity of the soil conditions.

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 according to Table 5-1. For loads associated with permanent removal after service life the load factors may be taken equal to 1.0.

4.2 Material factors

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 safety class
γ_m = 1.0 for the ALS for low and normal safety class
γ_m = 1.2 for the ALS for high safety class.

5 Free-Fall Anchors in Clay

5.1 General

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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5.1.2 Site-specific soil conditions shall be established in accordance with requirements set forth in DNV-OS-J101. For control of the installation, the anchor shall be instrumented.

Guidance note:

Double integration of accelerometer data can 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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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.

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 due to thixotropy and consolidation. Effects of cyclic loading on the final anchor resistance shall also be taken into account.

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 according to principles outlined in DNV-RP-E301.

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 and will account for the soil softening due to clay remoulding caused by the pile installation and the degree of clay reconsolidation at the time of hook-up to the floating structure. Design can also be carried out on the basis of instrumented tests on anchors, which are installed in the target area prior to the design phase, and which are not used for any other purpose.

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

5.1.8 Currently, no formal design procedure exists for free-fall anchors, but the design principles for piles as outlined in DNV-OS-J101 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.

5.1.9 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 shall be verified by measurements as required in [5.1.2].

5.1.10 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, as applicable, is initiated.

5.2 Material factors

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 safety class.

6 Fluke Anchors in Clay

6.1 General

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.

6.1.2 Fluke anchors are normally to be used only for unidirectional horizontal load application. A fluke anchor is therefore not suitable as a shared anchor point for more than one floating wind turbine unit. Some uplift may be allowed under certain conditions both during anchor installation and during operating design conditions. A recommended design procedure for fluke anchors is given in DNV-RP-E301.

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.

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.

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, reference is made to DNV-RP-E301.

6.2 Material factors

6.2.1 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 regardless of safety class
γ_m = 1.0 for the ALS for low and normal safety class
γ_m = 1.3 for the ALS for high safety class.

7 Plate Anchors in Clay

7.1 General

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.

7.1.2 Plate anchors are normally to be used only for unidirectional load application. A plate anchor is therefore not suitable as a shared anchor point for more than one floating wind turbine unit.

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 DNV-RP-E302.

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.

7.1.5 Recommended design procedures for plate anchors are given in DNV-RP-E302.

7.2 Material factors

7.2.1 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 safety class
γ_m = 1.0 for the ALS for low and normal safety class
γ_m = 1.3 for the ALS for high safety class.

8 Grouted Rock Anchors

8.1 General

8.1.1 Grouted rock anchors generally consist of steel elements, or tendons, such as steel bars or wire-rope strands, grouted in a pre-drilled hole in the rock. After grouting, the grout is subject to curing. The bars or strands are subsequently tensioned.

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 grouted rock anchors for a site in question.

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, permeability and cracks. Laboratory testing shall be used to quantify compressive and tensile strengths.

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.

8.1.5 For design of a grouted 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.

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.

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.

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.

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.

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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.

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.

8.1.11 Challenges involved with grouting and grouting procedures under water, in particular in deep waters, shall be considered.

Guidance note:

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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8.1.12 A post-installation test program shall be defined and carried out for each installed grouted rock anchor. 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 pretension level. The set of suitability tests should also include a creep test at the design pretension level. A proof test in which the anchor is subjected to posttension beyond the design pretension level shall also be carried out.

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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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 grouted 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.

SECTION 10 FLOATING STABILITY

1 Introduction

1.1 General requirements

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.

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 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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1.1.3 For permanently manned floating wind turbine units, sufficient floating stability is an absolute requirement. This applies to the operational phase as well as any temporary phases, and it applies to the intact floating unit as well as the unit in the damaged condition. For manned units, DNV-OS-C301, Stability and Watertight Integrity, can be applied for evaluation of both intact and damaged stability.

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 an absolute requirement in the intact condition. This applies to the operational phase as well as any temporary phases. For unmanned units in the damaged condition, sufficient floating stability is not a requirement, but an option which may be considered. Requirements for intact stability are given in [2]. Requirements for damaged stability, when damaged stability is opted for, are given in [3].

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.

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.

1.1.7 The stability requirements in [2] and [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 [2] and [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 according to specifications given in DNV-RP-C205, Sec.7.2.

1.1.8 Maximum vertical centre of gravity (VCG) limit curves shall be prepared according to the stability requirements specified in this section.

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.

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 the properties are to be obtained by inclination testing and units of the same design are manufactured at more than one yard, one inclination test shall be carried out at each yard.

1.1.11 For evaluation of sufficient floating stability it is crucial to assess the wind loads. The wind loads consist of rotor-filtered 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 DNV-OS-C301, Ch.2 Sec.1 B; 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, 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.

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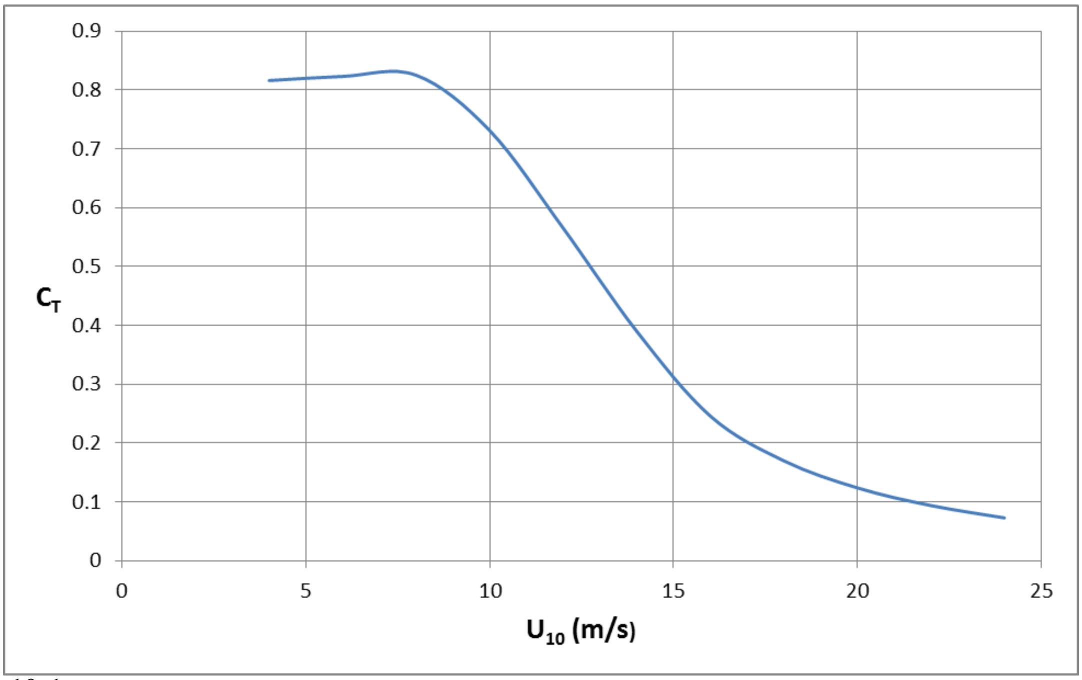

Figure 10-1
Thrust coefficient vs. 10-minute mean wind speed; rotor diameters 100-120 m

1.1.12 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. For an unmanned unit, 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.

1.1.13 Material requirements for permanent ballast used for stability purposes are given in Sec.6.

2 Intact Stability

2.1 General

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 due to the combined effects of thrust from the operating turbine and large waves.

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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 according to the methods specified in [1.1.11], assuming that the rotor plane is perpendicular to the direction of the wind.

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 according to the methods specified in [1.1.11], assuming that the rotor plane is parallel 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 in the fault situation that the turbine does not yaw out of the wind during severe storm conditions, it will be necessary to assume that the rotor plane is perpendicular to the direction of the wind when calculating the wind heeling moments; however, a wind speed of 36 m/s (70 knots) may be assumed for this situation.

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.

2.2 Ship-shaped structures

2.2.1 Requirements for intact stability of ship-shaped structures are given in [2.2.2].

2.2.2 The area under the righting moment curve to the second intercept or down-flooding 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.

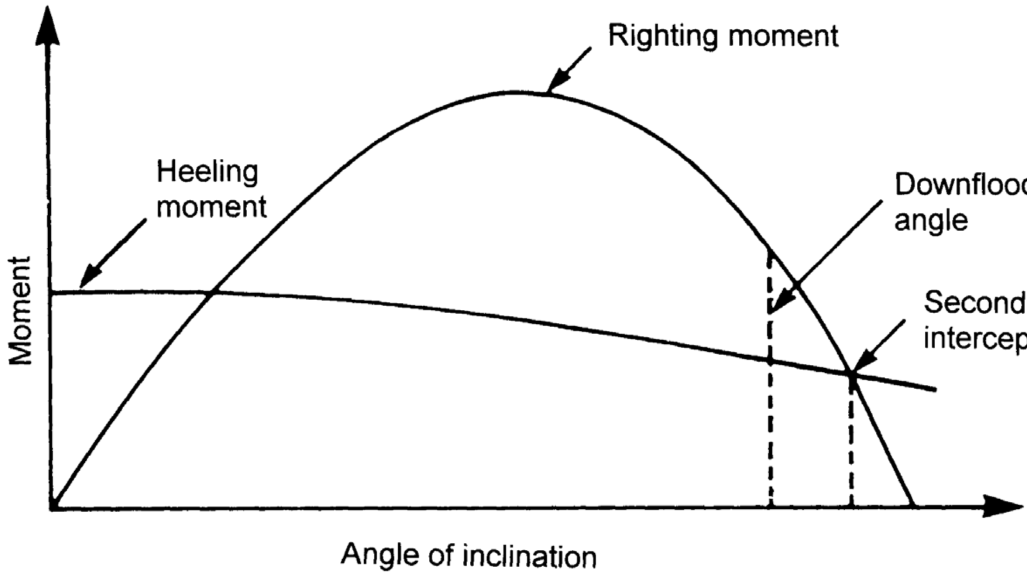

Figure 10-2
Righting moment and wind heeling moment curves

2.3 Column-stabilized structures

2.3.1 Requirements for intact stability of column-stabilized structures are given in [2.3.2] and [2.3.3].

2.3.2 For column-stabilized units such as semisubmersibles, the area under the righting moment curve to the angle of downflooding shall be equal to or greater than 130% of the area under the wind heeling moment curve to the same limiting angle.

2.3.3 The righting moment curve shall be positive over the entire range of angles from upright to the second intercept.

2.4 Deep Draught Floaters (DDFs)

2.4.1 Requirements for intact stability of DDFs are given in [2.4.2].

2.4.2 For deep draught floaters such as spars, 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.

2.5 Tension Leg Platforms (TLPs)

2.5.1 Requirements for intact stability of TLPs are given in [2.5.2].

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 column-stabilized units as defined in [2.3].

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, which is restrained in heave, roll and pitch, is typically provided by the pretension and stiffness of the tendon system, rather than by the water-plane area.

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 COG (Centre of Gravity) shift in various operational modes and environmental conditions.

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.

2.5.6 The allowable shift of COG may be presented as an envelope relative to the originally calculated COG.

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.

2.5.8 An inclination test or an analytical calculation as specified in [1.1.10] 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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3 Damaged Stability

3.1 General

3.1.1 For unmanned units, damaged stability is not a requirement, but an option which may be adhered to on a voluntary basis. Requirements for damaged stability given in this subsection apply to the case that damaged stability is opted for in design.

3.1.2 Additional compartmentalization as commonly used for manned structures, most often to provide for damaged 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 means 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 damaged stability is desirable.

The choice between multiple compartments and only one compartment in the floater hull structure can be based on a cost-benefit analysis. 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 be adequate for most unmanned units.

However, for large wind turbines in excess of approximately 10 MW, the cost associated with the loss of one unit due to 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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3.1.3 For assessment of stability in the damaged condition, it shall be demonstrated that the floating structure complies with the requirements of [3.2] to [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 be assumed that the rotor can be stopped and yawed out of the wind and that the blades can be feathered (pitched down). Requirements for assumptions regarding the extent of damage are given in [3.6].

3.2 Ship-shaped structures

3.2.1 Requirements for damaged stability of ship-shaped structures, when applicable, are given in [3.2.2] and [3.2.3].

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 [3.6].

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) superimposed from any direction. In this condition the final waterline, after flooding, should be below the lower edge of any downflooding opening.

3.3 Column-stabilized structures

3.3.1 Requirements for damaged stability of column-stabilized structures, when applicable, are given in [3.3.2] and [3.3.3].

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 can be made with or without inclusion of 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°.

3.3.3 When the buoyancy and the stability are assessed with inclusion of the wind heeling moment, it is recommended that the wind heeling moment be based on a wind speed of 25.8 m/s (50 knots) superimposed from any direction.

3.4 Deep Draught Floaters (DDFs)

3.4.1 Requirements for damaged stability of DDFs, when applicable, are given in [3.4.2] and [3.4.3].

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, after the damage set out above, shall have, from the first intercept to the lesser of the extent of weathertight integrity required by [3.4.2] (1) and the second intercept, a range of at least 7°.

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.

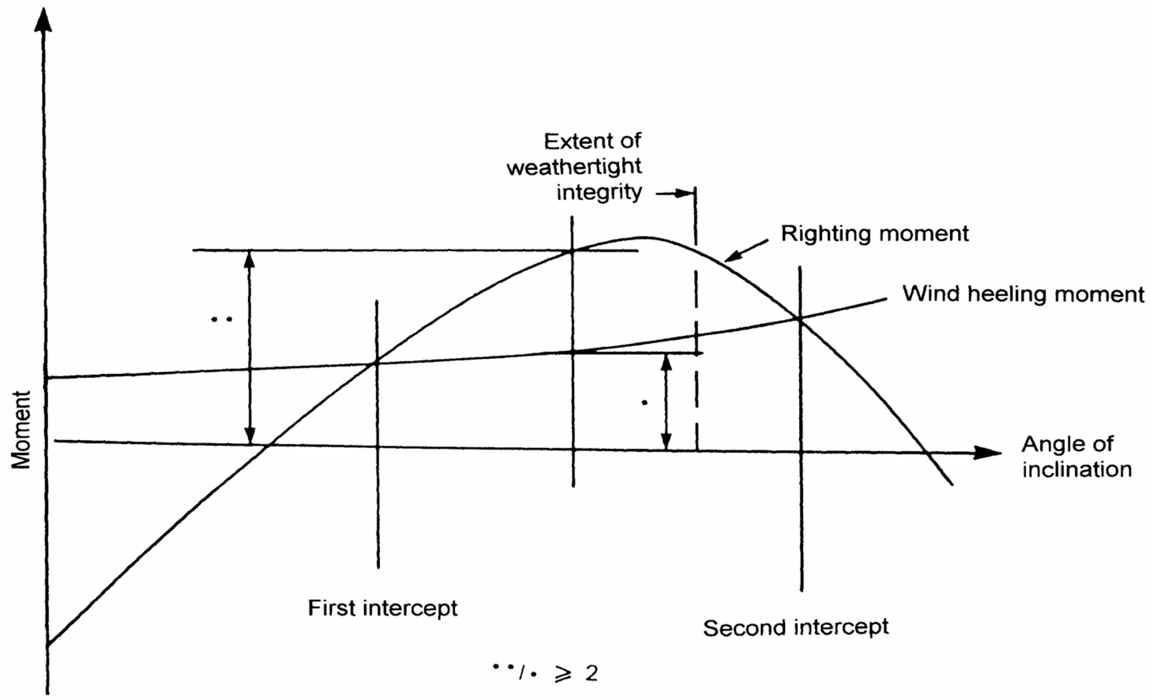

Figure 10-3
Righting moment and wind heeling moment curves

3.5 Tension Leg Platforms (TLPs)

3.5.1 Requirements for damaged stability of TLPs, when applicable, are given in [3.5.2] through [3.5.6].

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 column-stabilized units as defined in [3.3].

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 [2.5]. The allowable weight and the COG shift envelope shall be established for the damaged condition using the same procedure as defined in [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.

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.

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.

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.

3.6 Extent of damage

3.6.1 Assumptions for extent of damage are necessary in order to assess damage stability.

3.6.2 In assessing the damaged stability of floating units as specified in [3.2] through [3.5], 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.

4 Watertight Integrity

4.1 General requirements

4.1.1 Material requirements to support watertight integrity are given in Sec.6.

4.2 External openings

4.2.1 Watertightness of external openings for evaluation of intact stability shall be assessed according to DNV-OS-C301, Sec.2.

4.3 Internal openings

4.3.1 Internal openings need not be considered with respect to watertightness, unless damaged stability is opted for, see [1.1.4].

4.3.2 When damaged stability is opted for according to [1.1.4], 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.

4.4 Capacity of watertight doors and hatch covers; operation and control

4.4.1 Reference is made to DNV-OS-C301, Sec.2.

4.5 Load lines

4.5.1 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

1 Introduction

1.1 General

1.1.1 This section provides requirements for the floater motion control system. Requirements for other control systems such as the control and protection system for the wind turbine are not covered. However, it is a prerequisite that the wind turbine operation and safety are governed by a control and protection system as required by IEC 61400-1 and IEC 61400-3.

1.1.2 Requirements for instrumentation and control systems are addressed in DNV-OS-D202 and safety shutdown systems in DNV-OS-A101. DNV-OS-D202 and DNV-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.

1.1.3 The control and monitoring requirements for the wind turbine should be summarized in a Functional System Design (FSD) which describes the objectives and attributes of the control system in terms of functional capability at various locations such as on-board, electrical substation, and control centre. The FSD may be prepared using Sec.2 and Sec.3 of DNV-OS-D202 as a check list.

1.1.4 For the general installation of instrumentation equipment and systems in a wind turbine, the reference in DNV-OS-D202, App. F, contains aspects that should be considered. DNV-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 DNV-OS-D202 contents list that may provide relevant guidance is presented in App. F, F3 Instrumentation and Control Systems. This guidance will in many cases have to be modified or adapted to the particular wind turbine design under consideration.

2 Floater Motion Control

2.1 Background

2.1.1 Operation of the wind turbine at wind speeds above the rated wind speed implies that the wind turbine control system operates in the constant power regime, for example by use of blade pitch control. In this wind speed region there may be a negative gradient of thrust with respect to wind speed which will be felt as a negative damping in the surge and the pitch modes. This implies a situation where energy will be pumped into the motion of the floating structure. This may in turn lead to amplification of motion and response to more than what the structure may be able to withstand, unless an intervention against such amplification by means of a floater motion controller is implemented. Such intervention may be accomplished by means of the control system of the wind turbine and consists of adjustment of the blades to increase or decrease the rotor thrust as appropriate and thereby prevent excitation. The intervention may in principle also be accomplished by other control means such as pumping ballast back and forth or other suitable adjustment of ballast.

2.2 Floater motion controller

2.2.1 A specific floater motion controller shall be implemented to minimize excitation in the pitch mode of motion of the floating wind turbine structure and thereby stabilize the structure and minimize the structural responses. The specific floater motion controller can be based on a general conventional, constant-power control system for the wind turbine, such as a collective blade pitch control system. The floater motions can be measured and can then be used as an extra input signal to the conventional control system for the wind turbine.

Guidance note:
For a TLP, for which the pitch mode of motion is restrained, the floater motion control system is used to minimize the surge mode of motion.

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2.2.2 Considering the motions of a floating support structure as compared to a traditional, fixed support structure, some adjustments of the control system of the wind turbine may be necessary. This may give rise to special load cases. Such special load cases shall be identified and shall be accounted for in structural design. Special attention shall be given to the fact that larger inclinations might occur for floating wind turbines than for fixed wind turbines. This has an impact on systems such as the sensing systems and the activation systems.

2.2.3 When the floater motion control system and the wind turbine control system are integrated, the requirements for the wind turbine control system specified in IEC 61400-1 and IEC 61400-3 also apply to the integrated control system.

2.2.4 Possible failures of the floater motion control system shall be considered, in particular in the case that this control system is integrated with the wind turbine control system and implies that the wind turbine control system is modified.

2.3 Effect of floater motion control

2.3.1 The effect of the floater motion controller on the motion of the floating structure is expected to be more pronounced in less severe sea states where the loads on the floating structure are wind dominated. In more severe sea states with higher wave intensity, for which the loads on the floating structure are mainly wave dominated, the advantage of the controller is expected to be less significant.

3 Special Issues

3.1 Interaction with other systems

3.1.1 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.

SECTION 12 ELECTRICAL AND MECHANICAL SYSTEMS

1 Introduction

1.1 General

1.1.1 This section provides general requirements for various mechanical systems which are necessary for maintaining the normal operation of a floating wind turbine unit.

2 Mechanical Systems

2.1 General

2.1.1 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.

2.2 Bilge system

2.2.1 It shall be documented that bilge systems for removing and pumping bilge water serve the purpose. Bilge systems can be designed according to requirements given in DNV-OS-D101.

2.3 Ballast system

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.

2.3.2 Ballast systems can be designed according to requirements given in DNV-OS-D101.

2.4 Compartment venting

2.4.1 Compartments and tanks that are equipped with fixed means of drainage shall also be equipped with a ventilation arrangement.

2.5 Mooring equipment

2.5.1 Mooring equipment, such as winches and windlasses, chain stoppers, fairleads and systems to tension the mooring lines, can be designed according to structural design procedures specified in DNV-OS-E301 Ch.2 Sec.4. The design of these components is then based on a load equal to the characteristic breaking strength of the mooring lines. In this context, DNV-OS-E301 Ch.2 Sec.2 provides information about the minimum breaking load and other aspects of importance for the design.

2.6 Cranes

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 according to DNV Standard for Certification 2.22 Lifting Appliances.

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 due to the motions of the floating units.

2.6.3 Special attention should be given to dynamic amplification factors for lifting of wind turbine components.

2.7 Installation systems and pull-in systems

2.7.1 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.

2.8 Turret systems

2.8.1 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.

2.9 Power generation systems

2.9.1 Power generation systems for power supply for winches and other equipment shall be installed.

2.10 Fire fighting systems and equipment

2.10.1 Fire fighting systems and equipment shall be designed under due consideration of the size, type and intended service of the floating wind turbine unit. DNV-OS-A101 provides a general description of the safety philosophy for fire fighting systems and equipment.

2.11 Other equipment

2.11.1 Mechanical systems for tightening of tendons, for example following long-term creep and relaxation, shall be installed on floating units supported by tendons.

3 Electrical Systems

3.1 Lightning and earthing system

3.1.1 A lightning and earthing system shall be in place. The lightning and earthing system shall be designed according to the requirements of DNV-OS-D201, Sec.2, items I600 and I700, and to the requirements of IEC 61892-6, clauses 4 and 16.

SECTION 13 CORROSION PROTECTION

1 Introduction

1.1 General

1.1.1 The requirements for corrosion control given in DNV-OS-J101 apply to floating wind turbine structures with the exceptions, deviations and additions specified in this section.

1.2 Splash zone

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.

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 DNV-OS-J101. The definition of the splash zone for external surfaces of a floating support structure is given in [1.2.3] and [1.2.4]. The definition of the splash zone for internal surfaces of a floating support structure is given in [1.2.5].

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 vertical downwards motion of the floating structure, 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
— the vertical upwards motion of the floating structure, if applicable.

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 DNV-OS-J101.

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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 DNV-OS-J101, Sec.13.

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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1.2.5 The upper limit of the internal splash zone is the highest operational ballast water level in the compartment in question, increased by

— the largest upwards movement of ballast water owing to sloshing, if applicable.

The lower limit of the internal splash zone is the lowest operational ballast water level in the compartment in question, reduced by

— the largest downwards movement of ballast water owing to sloshing, if applicable.

1.3 Other issues

1.3.1 Requirements for corrosion allowance in internal compartments, which are not corrosion protected by coating, are specified in DNV-OS-J101. For internal compartments which are filled with solid ballast, seawater and/or air and which are permanently sealed (i.e. no plan for periodical inspections), these requirements for corrosion allowance can be waived in favour of no corrosion allowance. It is a prerequisite that the solid ballast does not contain sulphides. Also for internal compartments which are filled with air and which are accessible for inspection and repair, the requirements for corrosion allowance can be waived in favour of no corrosion allowance, if the relative humidity in the compartment can be controlled to be 80% maximum.

Guidance note:

Climate control to keep the relative humidity below a specified threshold is not common in unmanned offshore structures.

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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 type 1 1)	*Regular inspection type 2* 2)
Splash zone	0.4	0.2
Catenary 3)	0.3	0.2
Bottom 4)	0.4	0.3

1) The regular inspection is carried out by ROV according to DNV-OSS-102 Ch.3 Sec.6 B800 or according to operator's own inspection program, approved by national authorities if necessary. The mooring line has to be replaced when the diameter of the chain with the allowable breaking strength used in design of the mooring system, taking into account corrosion allowance, is reduced by 2%.

2) The regular inspection is carried out according to DNV-OSS-102 Ch.3 Sec.6 B700 or according to operator's own inspection program, approved by national authorities if necessary. The mooring line has to be replaced when the diameter of the chain with the allowable breaking strength used in design of the mooring system, after the corrosion allowance has been consumed, is reduced by 2%.

3) Suspended length of mooring line below the splash zone and always above the touchdown point.

4) Significantly larger corrosion allowance than the minimum values recommended in the table should be considered if bacterial corrosion can be expected.

1.3.3 The influence from energized power cables on corrosion and corrosion rates on the structural interface of the floater unit locally at the attachment point shall be considered when the corrosion protection of this unit is designed.

SECTION 14 TRANSPORT AND INSTALLATION

1 Marine Operations

1.1 General

1.1.1 Unless otherwise specified, the requirements for planning and execution of marine operations for transport and installation given in DNV-OS-J101 apply.

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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1.1.3 DNV-RP-H103 provides guidance for modelling and analysis of marine operations.

2 Risk Management during Marine Operations

2.1 Risk management in transportation and installation phases

2.1.1 DNV-RP-H101 provides guidance on how to manage and mitigate risks related to marine operations.

3 Marine Warranty Surveys

3.1 General

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.

3.1.2 DNV-OS-J101 provides guidance for marine warranty surveys of marine operations for transport and installation of offshore wind farms. For further details, reference is made to DNV Rules for Planning and Execution of Marine Operations.

4 Marine Operations – General Requirements

4.1 General

4.1.1 Requirements and recommendations relevant for marine operations during temporary phases for wind turbine structures are given in DNV-OS-J101 and are applicable to marine operations during temporary phases for floating wind turbine structures.

4.1.2 The requirements for vessel stability given in DNV-OS-J101 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.

4.1.3 Requirements for stability of floating wind turbine structures during transport and installation are given in Sec.10.

5 Marine Operations – Specific Requirements

5.1 Load transfer operations

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.

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.

5.1.3 Requirements for load transfer operations are given in DNV-OS-H201.

5.2 Sea transports

5.2.1 Specific requirements and guidelines for single-vessel and barge-towing operations are given in DNV 'Rules for Planning and Execution of Marine Operations', Part 2, Chapter 2.

5.2.2 Requirements and recommendations for transport on-board ship, towing of multi-hull vessels, self-floating and self-propelled carrier transports are given in DNV 'Rules for Planning and Execution of Marine Operations', Part 2, Chapter 3.

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.

5.3 Offshore installation

5.3.1 Specific requirements and recommendations for offshore installation operations applicable for wind turbines and their support structures and structural components are given in DNV 'Rules for Planning and Execution of Marine Operations', Part 2, Chapter 4.

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.

5.3.3 Operational aspects for ballasting and grouting shall be considered.

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.

5.4 Lifting operations

5.4.1 Guidance and recommendations for lifting operations, onshore, inshore and offshore, of objects with weight exceeding 50 tonnes are given in DNV 'Rules for Planning and Execution of Marine Operations', Part 2, Chapter 5.

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.

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.

5.5 Subsea operations

5.5.1 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 DNV 'Rules for Planning and Execution of Marine Operations', Part 2, Chapter 6.

SECTION 15 IN-SERVICE INSPECTION, MAINTENANCE AND MONITORING

1 Introduction

1.1 General

1.1.1 The provisions set forth in DNV-OS-J101 for in-service inspection, maintenance and monitoring shall apply.

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.

1.2 Fibre ropes, tethers and tendons made from synthetic fibre yarns

1.2.1 The provisions set forth in DNV-OS-E303 for in-service inspection, maintenance and monitoring shall apply.

1.2.2 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

1 Introduction

1.1 General

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.

1.1.2 This section makes reference to a number of available design codes of relevance for power cable design. Normative references are given in cases where requirements to be fulfilled are specified in the referenced codes. Where requirements specified in this section conflict with those of referenced standards, the provisions of this section shall prevail. Informative references are given in cases where useful guidance can be found in the referenced codes.

1.1.3 Wherever normative references to other codes are made, the provisions of these other codes shall be adhered to in design.

2 References

2.1 Overview of relevant standards

2.1.1 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 including the cable–floater interface.

2.1.2 Although most 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 codes for analysis and design of dynamic power cables

Code	*Title*	*Design aspects*
ISO 13628-5	Subsea umbilicals	Main reference for mechanical design of dynamic power cables, providing requirements for global load effect analyses and requirements for local load effect analyses.
DNV Rules	Planning and Execution of Marine Operations	See guidance note below.
DNV-OS-H101	Marine Operations, General	General requirements and recommendations for planning, preparations and performance of marine operations.
DNV-OS-H102	Marine Operations, Design and Fabrication	General requirements and recommendations for selection of loads, design (verification) and fabrication of structures involved in marine operations.
DNV-OS-F201	Dynamic risers	Analysis guidance. Outline of global response model verification. Guidance on statistical response processing.
DNV-RP-C203	Fatigue Design of Offshore Steel Structures	Fatigue design capacity curves of standard materials.
DNV-RP-C205	Environmental conditions and environmental loads	Specification of environmental loading, choice of hydrodynamic coefficients etc., and principles for floater motion analysis.
DNV-RP-F203	Riser Interference	Principles for assessment of riser interference.
DNV-RP-F204	Riser Fatigue	Principles for riser fatigue assessment and simplified VIV analysis guidance.
DNV-RP-F205	Global Performance Analysis of Deepwater Floating Structures	Guidance on floater motion and station-keeping analysis.
DNV-RP-F401	Electrical Power Cables in Subsea Applications	Supplement to ISO 13628-5, covering subsea power cables. Design and acceptance criteria for power cables, cable components and cable terminations.
API Spec. 17J	Specification for unbonded flexible pipe	Acceptance criteria for tensile armour. Acceptance criteria for polymer layers in flexible pipes.
API Spec. 17L1	Specification of flexible pipe ancillary equipment	Design guidance for ancillary components such as buoyancy modules, bend stiffeners etc.

Guidance note:

DNV Rules for Planning and Execution of Marine Operations is currently being restructured, and will be replaced by the following Offshore Standards (OS):

- DNV-OS-H101, *Marine Operations, General*
- DNV-OS-H102, *Marine Operations, Design & Fabrication*
- DNV-OS-H201, *Load Transfer Operations*
- DNV-OS-H202, *Sea Transports*
- DNV-OS-H203, *Transit and Positioning of Mobile Offshore Units*
- DNV-OS-H204, *Offshore Installation Operations*
- DNV-OS-H205, *Lifting Operations*
- DNV-OS-H206, *Sub Sea Operations*

Each OS will enter into force at the date of publication. Until the OS is published the relevant requirements in "DNV Rules for Planning and Execution of Marine Operations" shall be considered governing.

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3 Design Principles

3.1 General

3.1.1 The power cable system, including its interface with the floater, shall be designed according to 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.

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.

3.1.3 The power cable system shall be designed according to DNV-RP-F401 and ISO 13628-5, as far as these two standards are applicable.

3.1.4 A design basis, containing or referring all relevant requirements to the power cable system, shall be established.

3.1.5 A design methodology, including at 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.

3.1.6 The power cable system shall be designed to meet the functional requirements specified in the design basis.

3.2 Design principles

3.2.1 The power cable with end terminations 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 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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3.2.2 The partial safety factor method is often applied in design of steel structures such as cable terminations and other cable ancillaries. 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.

3.2.3 ISO 13628-5 and DNV-RP-F401 apply the Working (allowable) Stress Design (WSD) method for design of umbilicals and power cables, respectively. 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. Reference is made to DNV-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 method 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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3.2.4 The power cable system shall be checked for the following limit states during both installation and operation:

— Ultimate Limit State (ULS)
— Accidental Limit State (ALS)
— Fatigue Limit State (FLS).

For details about these limit states and how they are defined, see Sec.1 and Sec.2.

4 Functional Requirements

4.1 Power cable

4.1.1 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, at a minimum, in order 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
— function as intended for the duration of its intended service life.

4.1.2 The following shall, at a minimum, be considered in design of the power cable:

— wear
— fretting
— corrosion
— dimensional tolerances
— material creep
— ageing.

4.2 Pull-in arrangement

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.

4.2.2 The pull-in arrangement shall ensure a safe transfer of cable tension to the pull-in wire while maintaining the functionality of the power cable and ancillary equipment such as hang-off termination, floater connector, bend stiffener or similar.

4.3 Hang-off termination

4.3.1 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 ancillary 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.

4.4 Bend stiffener/bell-mouth

4.4.1 A bend stiffener or bell-mouth at the floater interface may be applied in order to reduce local bending stress in the cable components and distribute fatigue damage over the length of the bend stiffener.

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.

4.5 Floater structural interface

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 in order to simplify connection of a cable or its bend stiffener at the floater interface.

4.5.2 The connector arrangement shall be considered as part of the floater structure and shall be designed according to Sec.7.

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.

4.5.4 The connector part assembled to the power cable during installation shall be designed for relevant installation loads according to [6].

5 Analysis Methodology

5.1 Power cable structural capacities

5.1.1 Accounting for design resistance, addressed in [8.1] to [8.4], the following shall be established (at a minimum):

— cable capacity in combined tension and bending, ref. capacity curve concept as defined in ISO 13628-5
— cable capacity with respect to radial compression from tensioner tracks and chute contact at maximum cable installation load.

5.1.2 The cable design documentation should clearly state how much of the cable's fatigue life that is attributed the installation and operation phases, respectively.

5.1.3 As a general rule, negative effective tension, i.e. compressive axial load, in the power cable should be avoided.

Guidance note:
Negative effective cable tension may be acceptable, provided that an FMECA has been carried out and that analyses and/or tests have been performed to demonstrate the cable's robustness with respect to all relevant failure modes.

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5.2 Installation analyses

5.2.1 All phases of the cable installation operation shall be analysed according to the principles stated in DNV Rules for Planning and Execution of Marine Operations. See also guidance note to Table 16-1.

5.2.2 For temporary operational conditions the load effect return period for definition of characteristic environmental loads 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, reference is made to Table 4-1 and Table 4-2. For details about how to calculate characteristic loads from characteristic conditions, reference is made to DNV-OS-H101.

5.2.3 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 DNV-OS-H101, 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 DNV-OS-H101.

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5.2.4 The cable installation analyses shall establish 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 [5.2].

5.2.5 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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5.3 Global (in-service) analyses

5.3.1 For permanent operational conditions, a 50-year return period ($2 \cdot 10^{-2}$ annual exceedance probability) applies for definition of characteristic environmental loads.

5.3.2 Global load effect analyses of the power cable system shall be performed according to DNV-OS-F201.

5.3.3 Fatigue analysis of the power cable system shall consider all relevant cyclic load effects, as described in DNV-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)

5.3.4 The effect of VIV on the suspended power cable shall be considered, ref. ISO 13628-5.

5.3.5 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
— drag and inertia effects
— structural damping.

5.3.6 The estimation of hydrodynamic load on power cables subjected to accumulated marine growth shall account for the increase in effective diameter and surface roughness. DNV-RP-C205 may be used for guidance.

5.3.7 Sensitivity studies shall be performed in order to ensure that the system configuration is robust. Sensitivity with respect to the following shall be assessed (at a minimum):

— marine growth
— drag and inertia effects
— structural damping
— current
— soil data
— floater draft and corresponding motions (i.e. transfer functions)
— cable installation tolerances (e.g. location of cable touchdown point etc.).

6 Loads and Load Effects

6.1 Loads

6.1.1 Functional, environmental and accidental loads are defined in Sec.4.

6.1.2 Characteristic loads are defined in Sec.4. These definitions apply both to the power cable and to the cable ancillaries.

6.1.3 For power cables, characteristic loads applicable to the installation phase shall be determined according to Sec.4.

6.1.4 For cable ancillaries, characteristic loads applicable to the installation phase shall be determined according to DNV-OS-H102.

6.1.5 Characteristic loads applicable to the operational phase shall be determined according to Sec.4. This applies both to the power cable and to the cable ancillaries.

6.2 Design load effects for power cables

6.2.1 The design load effect S_d for a power cable is equal to the sum of unfactored characteristic load effect contributions from functional loads, environmental loads and/or accidental loads, as applicable. Note that different design load effects apply to the installation phase and to the operational phase.

6.3 Design load effects for cable ancillaries of steel

6.3.1 Pull-in heads shall be regarded as lifting equipment during installation and shall be designed for relevant installation loads accordingly. Similarly, cable terminations and other ancillaries supporting the weight of a suspended cable during installation shall be regarded as lifting equipment and designed for relevant installation loads accordingly.

Guidance note:

According to ISO 13628-5, a pull-in head is regarded as lifting equipment during installation.

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6.3.2 Cable ancillaries regarded as lifting equipment shall be designed according to DNV Rules for Planning and Execution of Marine Operations, Pt. 2, Ch. 5, *Lifting* (to be replaced by *DNV-OS-H205*). These rules require the application of a consequence factor, γ_c, to the characteristic lifting load, in addition to the load factor, γ_f, given in DNV-OS-H102. Hence, the design lifting load F_d shall be calculated as

$$F_d = \gamma_f \cdot \gamma_c \cdot F_c$$

in which F_c is the characteristic lifting load, defined as a characteristic permanent load or a characteristic variable functional load, as applicable.

6.3.3 The load factors and consequence factors specified in Table 16-2 shall be applied in design of cable ancillaries of steel in the installation phase.

Table 16-2 Load factors and consequence factors, lifting

Element category	γ_f	γ_c
Lift points including attachments to object (single critical elements supporting the lift points are defined within this category)	1.3	1.3
Lifting equipment (e.g. spreader frames or beams, plate shackles)	1.3	1.3
Main elements supporting the lift point	1.3	1.15
Other elements of lifted object	1.3	1.0

NOTES

1) γ_f = load factor

2) γ_c = consequence factor

3) γ_c is meant to account for severe consequences of single element failure. Categorization of elements according to the above should hence duly consider redundancy of elements.

6.3.4 Cable ancillaries, which are not regarded as lifting equipment during installation, shall be designed for relevant loads during installation. Likewise, cable ancillaries shall be designed for relevant loads during operational conditions. In both cases, the design load effects are obtained by multiplying the characteristic load effect of each load category by its corresponding load factor and superimposing the contributions, for example

$$S_d = \gamma_F \cdot S_{eF} + \gamma_E \cdot S_{eE} + \gamma_A \cdot S_{eA}$$

in the case that the load effect of interest is the stress in a particular component, where

S_{eF} = characteristic stress from functional loads
S_{eE} = characteristic stress from environmental loads
S_{eA} = characteristic stress from accidental loads

and γ_F, γ_E and γ_A are associated load factors.

Guidance note:

As the load effect, e.g. stress resulting from an applied load, is not necessarily linear with the applied load, the specified load factors are applied to the characteristic load effects and not directly to the applied characteristic loads. For details regarding when load factors are applied to load effects and when they are applied to loads, reference is made to Sec.2.

Different characteristic environmental load effects apply during installation than during operational conditions; see Table 4-1 and Table 4-2. Characteristic environmental load effects during installation depend on conditions such as the duration of the installation; see DNV-OS-H101.

Load combinations to be checked are specified in Sec.5.

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6.3.5 The design of cable ancillaries of steel against the ULS and the ALS in the operational condition shall be carried out according to requirements given in Sec.7, based on load combinations and safety factor requirements given in Sec.5 for design to normal safety class.

6.4 Design load history for fatigue assessment

6.4.1 The design load history for fatigue assessment and fatigue design of power cables and cable ancillaries shall be taken as the expected load or stress history over the design life of the cable component in question.

7 Material Strength

7.1 General

7.1.1 Requirements for typical cable components are given in DNV-RP-F401.

7.1.2 Material strength for cable components of steel is represented by SMYS and SMTS.

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 due to temperature effects are given in DNV-OS-F201.

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8 Design Resistance and Design Criteria

8.1 Characteristic resistance

8.1.1 The characteristic strength of steel shall be taken as the smaller of

— SMYS adjusted for temperature derating
— 90% of SMTS adjusted for temperature derating.

8.1.2 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 ancillaries.

8.2 Resistance factors

8.2.1 For design of the power cable the utilization factors for load-carrying cable elements shall not be taken greater than the values specified in Table 16-3.

Table 16-3 Utilization factors for steel armour

	Utilization factor, η
Normal operation (ULS)	0.62
Installation	0.78
Abnormal operation (ALS)	1.00

8.2.2 For design of cable ancillaries with respect to lifting loads during installation, the material factor shall not be taken less than γ_m = 1.15 in the ULS and the ALS.

8.2.3 For design of cable ancillaries in the operational condition, the material factor shall not be taken less than γ_m = 1.10 in the ULS and the ALS.

8.3 Design criteria – general

8.3.1 Principles for establishing acceptance criteria for the power cable, the cable components and cable terminations are given in DNV-RP-F401.

8.4 Ultimate and accidental limit states

8.4.1 For the power cable, the general design criterion for load bearing steel components is

$$S_d \leq \eta \cdot R_k$$

in which S_d is the design load effect, η is the utilization factor as given in [8.2], and R_k is the characteristic resistance as calculated from the characteristic strength and the nominal properties of the structural component.

8.4.2 For cable ancillaries regarded as lifting equipment during installation, the design criterion is

$$F_d \leq \frac{R_k}{\gamma_m}$$

in which F_d is the design lifting load as calculated in [6.3.2].

8.4.3 For cable terminations and other ancillaries of steel, the design criterion in the operational phase is

$$S_d(S_{kF}, S_{kE}, S_{kA}, \gamma_F, \gamma_E, \gamma_A) \leq \frac{R_k}{\gamma_m}$$

in which S_d is the design load effect calculated as detailed in [6.3] and in Sec.2 and R_k is the characteristic resistance as calculated from the characteristic strength and the nominal properties of the cable. γ_m is the material resistance factor given in [8.2].

8.5 Fatigue limit state

8.5.1 The power cable shall be qualified with respect to fatigue as specified in DNV-RP-F401.

8.5.2 Armour terminations subjected to significant fatigue loading shall be qualified with respect to fatigue strength by testing as specified in DNV-RP-F401.

8.5.3 Calculation of fatigue stresses shall consider stick/slip behaviour of the cable components, as well as dimensional effects of wear and corrosion.

8.5.4 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.

8.5.5 For steel components, *DFF* shall not be taken less than 10 unless otherwise agreed. Reference is made to DNV-RP-F401.

8.5.6 The design criterion in fatigue is

$$D_D \leq 1.0.$$

9 Other Issues

9.1 Interaction with fishing equipment

9.1.1 Possible interaction with fishing equipment, such as trawls, shall be considered in design.

9.2 On-bottom stability

9.2.1 On-bottom stability of power cables resting on the seabed shall be addressed in the cable design. DNV-RP-F109 can be used for on-bottom stability design of power cables resting on the seabed.

9.3 Corrosion

9.3.1 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.

9.3.2 Electrical continuity or electrical isolation, as required, between the cable components, hang-off termination and floater structure shall be ensured considering the overall system design.

9.4 Earthing of lightning conductors

9.4.1 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.

9.5 Protection against mechanical damage

9.5.1 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. Reference is made to DNV-RP-F107, which provides guidance on which types of protection are suitable for different purposes.

9.6 Redundancy

9.6.1 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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APPENDIX A ANALYSIS GUIDANCE

A.1 Introduction and Objective

A.1.1 General

A.1.1.1 The objective of this appendix is to provide useful guidance for execution of coupled analysis of a wind turbine and its floating support structure including the station keeping system.

A.1.1.2 The term “coupled analysis” used throughout this appendix means that the structural dynamic responses from the aerodynamics and hydrodynamics loads are coupled. No coupling effects between the models of the external conditions are considered.

A.1.1.3 Over the past 10 years, several numerical studies, model tests and prototype tests have demonstrated the technical feasibility of floating wind turbines. However they also have revealed the need for integrated analysis, comprising the experience accumulated by both the wind energy industry and the O&G industry.

A.1.1.4 Several studies have shown that while the power performance of a floating wind turbine are dominated by the aerodynamics of the wind turbine rotor, the loads on a floating wind turbine are driven by the floater displacements, see (Jonkman and Matha, 2010) and (Henderson and Patel, 2003). In another study, (Larsen and Hanson, 2007) show the relevance of the interaction between the wind turbine control system and the floater natural modes for the dynamic stability of the system. These studies are just two examples of the strong coupling of the hydrodynamics and aerodynamics in the floating wind turbine design and analysis.

Guidance note:

Modelling of wind turbine control system faults may be rather important with a view to the impact that such faults can have for floating wind turbine units, e.g. regarding the stability of such units in fault conditions.

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A.1.1.5 Moreover the experience accumulated in the past in the wind energy and O&G industries have followed different methodologies. For example, the O&G numerical studies of an offshore platform have commonly been carried out in the frequency domain, while it has been common to analyse the wind turbine in the time domain. In recent years, however, this has changed as time domain analysis has become more common also in the O&G industry. Other typical differences include: different typical periods of the structure (much larger in case of offshore O&G platforms than in case of wind turbines), different time scale of simulations and sampling, different terminology, different typical size of the structures to analyse.

A.1.1.6 This appendix aggregates experience accumulated by the wind industry and the O&G industry, in order to obtain a set of recommendations for the analysis of an integrated floating wind turbine system. The experience from studies by universities, laboratories and national institutes is also included in the appendix.

A.1.1.7 The recommendations are based on the current state of the art of the technology. Since the concept of floating wind turbines can be seen as a quite novel technology, new developments will take place and new experience will be gained over the next few years and consequently recommendations that can be given may be expected to change in process of time.

A.1.2 Assumptions

A.1.2.1 The following assumptions and definitions are made:

— The models described and recommended in this appendix are based on the state of the art knowledge. No new method or model is introduced.
— There is not yet one unique design of a floating wind turbine. The appendix is based on the state of the art 2013. Some recommendations are given for some specific concepts. However the details and set up of a coupled FWT analysis have to be evaluated on the specific case.
— The design and verification of a Floating Wind Turbine (FWT) is largely dependent on the specific site. Some recommendations are given for some known site-related issues. However specific issues related to a specific site assessment shall be evaluated separately.

A.1.3 Methodology

A.1.3.1 The analysis of a FWT can be facilitated by dividing the system in different sub-components.

A.1.3.2 Three subcomponents are identified:

— Wind Turbine (WT)
— Floating support structure, here referred to as “floater” (FL), including tower
— Station Keeping System (SKS), including the anchors.

Guidance note:

The tower is here grouped with the floating support structure rather than with the wind turbine. However, it is an option to group the tower with the turbine instead of with the floater, since it can be argued that the tower has some interaction with the aerodynamic model and the tower stiffness may influence the aerodynamic loads on the wind turbine. If the coupled model is assembled properly, it does not matter where the interface between the wind turbine and the floating support structure is located – at the tower top or the tower base. Most aero-elastic tools that have been adapted for floating wind turbine applications have the interface placed at the tower base.

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A.1.3.3 It is emphasized that particular care should be put in modelling the interfaces between the subcomponents (see [A.5.9]).

A.1.3.4 Different levels of accuracy are possible in modelling each of the subcomponents and the interfaces to couple them. Three accuracy levels are defined for the models of the subcomponents (see Subsection [A.5]).

A.1.3.5 The environmental conditions (wind, waves and currents) are defined in Table A-2 and the models for reproduce them are described in Subsection [A.3].

A.1.3.6 Based on the accuracy levels, an accuracy matrix is created, as reported in Table A-1. It is recommended that for the analysis of each subcomponent, the environmental conditions and the other subcomponents are modelled according to this table.

Table A-1 Model accuracy matrix

Component to analyse	*Wind*	*Waves*	*Currents*	*WT*	*FL*	*SKS*
FWT	Turbulent (Table A-2)	Linear (Table A-2)	Stationary	Level II (see [A.5.1])	Level II (see [A.5.2])	Level II/ LevelIII 1) (see [A.5.3])
WT	Turbulent (Table A-2)	Linear (Table A-2)	Can be omitted1	Level I (see [A.5.1])	Level III (see [A.5.2])	Level III (see [A.5.3])
FL	Deterministic/ Turbulent 2) (Table A-2)	Linear 3) (Table A-2)	Stationary / Unsteady4	Level III (see [A.5.1])	Level I (see [A.5.2])	Level I (see [A.5.3])
SKS	Deterministic/ Turbulent (Table A-2)	Linear 3) (Table A-2)	Unsteady	Level III (see [A.5.1])	Level II (see [A.5.2])	Level I (see [A.5.3])

1) Simplifications are possible depending on the FWT concept and the site assessment.
2) For a first stage analysis deterministic wind could be used. Turbulent wind shall be used when more detailed analysis is required, for example in the case of large asymmetric floaters (FWT).
3) It is recommended that the wave theory applied is adjusted to the particular stage in the design process. For an initial assessment or screening it may be sufficient to apply Airy (linear) wave theory for most cases, whereas for detailed design the wave theory should be chosen to be applicable with the governing design conditions.
4) The accuracy level has to be evaluated according to the site and to the floater concept (see [A.3.3]).

A.1.3.7 Depending on the specific analysis, a specific accuracy level will be required, thus specific software will be needed.

A.1.3.8 It is emphasized that the recommendations in Table A-1 are general recommendations: specific issues related to the concept to analyse and the specific site assessment have always to be critically considered. In order to facilitate this process, some suggestions from past experience are highlighted (see Subsection [A.6]) for some of the most relevant concepts.

A.2 Definitions of Environmental Conditions and Subcomponents

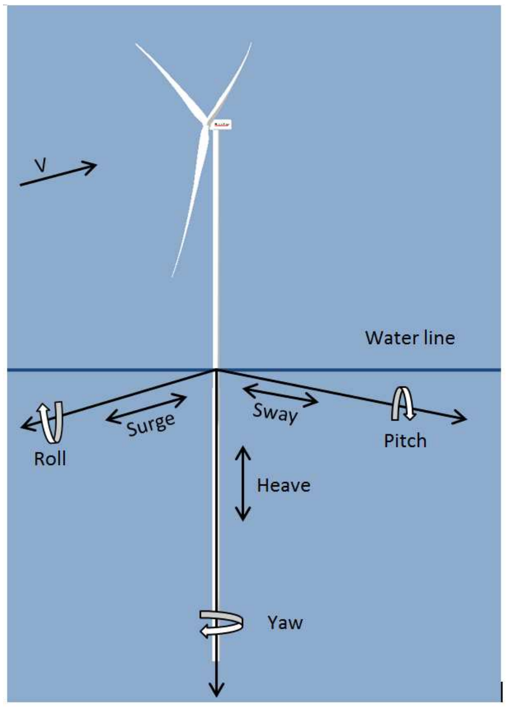
A.2.1 General

A.2.1.1 For terminology and definitions, reference is made to Sec.1 and to DNV-OS-J101.

A.2.1.2 In particular, a floating wind turbine (FWT) is here defined as “the entire system consisting of wind turbine, floating support structure and station keeping system”, also denoted as “floating wind turbine unit”.

A.2.2 Degrees of freedom

A.2.2.1 In order to analyse the full system, a reference system is considered as in [A.2.3], with origin in the still water level and one axis directed in the direction of the mean wind speed. Six degrees of freedom (DOF), correspondig to the six rigid-body modes of motions, are defined as in [A.2.3].



Surge	Translation along the longitudinal axis (main wind direction)
Sway	Translation along the lateral axis (transversal to the main wind direction)
Heave	Translation along the vertical axis
Roll	Rotation about the longitudinal axis
Pitch	Rotation about the lateral axis
Yaw	Rotation about the vertical axis

Figure A-1
DOFs of a floating wind turbine

A.2.3 Environmental conditions – general

A.2.3.1 The environmental conditions are defined as given in Table A-2.

Table A-2 References for definitions of environmental conditions

	Type	*Notes*	*Reference*
Wind	Turbulent (stochastic) model	— Several power spectral density models are available, see IEC 61400-1, DNV-OS-J101 and DNV-RP-C205 — The “Mann model” provides a method to generate turbulence spectra such as the Kaimal spectrum — Ideal time scale is 10 min with a view to validity of stationarity assumption for wind — Longer time scales than 10 min may be necessary for analysis of floating support units	DNV-OS-J101
	Deterministic wind		DNV-OS-J101
Waves	Linear (Stochastic and deterministic)	— Airy theory, Stream theory (deterministic non-linear wave model) or a comparably accurate theory — Corrections are recommended for shallow waters (Wheeler stretching correction) — Typical time scales for simulation of waves are in the range 3 to 6 hours with a view to validity of stationarity assumption	DNV-RP-C205
	Theories including second order effects or superior	— The theory should be selected evaluating the site environmental conditions and the concept	DNV-RP-C205
Currents	Stationary	— Variation with the depth shall be included according to the site	DNV-RP-C205
	Unsteady		DNV-RP-C205
Water level	Tide		DNV-RP-C205
	Storm surge		DNV-RP-C205

A.2.4 Wind

A.2.4.1 For a wind turbine the wind regime can be categorized into:

— Normal wind conditions. These typically are governing for the fatigue loads on the wind turbine. Severe sea-state conditions combined with normal wind conditions can drive wind turbine loads in some specific environments such as shallow water.
— Extreme wind conditions. These typically are governing for extreme loads on the wind turbine. The extreme wind loads may also contribute with non-negligible fatigue loads.

A.2.4.2 The selection of the proper wind speed and direction shall be carried out according to the standards DNV-OS-J101 and IEC 61400-1 and according to the load case to be simulated.

A.2.5 Waves

A.2.5.1 Waves are irregular and random in shape, height and speed of propagation. Such an irregular sea state is typically described by a frequency spectrum (see references in Table A-2) defined in terms of the significant wave height, the peak wave period, the main wave direction and some spreading function. Typically a real sea state can consist of several such irregular sea states. Sea states can be classified into:

— Wind seas. These are generated by the local wind conditions, and are typically irregular and short-crested.
— Swell seas. These have no relation to the local wind conditions, and are typically long (in terms of wavelength) and more regular and long-crested than wind sea waves.

A.2.6 Currents

A.2.6.1 The importance of the currents for analysis of floating wind turbines depends on the concept to be modelled. From O&G experience, currents are especially relevant for spar systems.

A.2.6.2 Currents may be categorized into:

— wind induced currents
— tidal currents
— storm surge currents
— ocean currents (geostrophical currents)
— loop and eddy currents
— coastal currents (longshore currents).

A.2.6.3 Wind generated and tidal currents are usually dominant and shall always be evaluated, while the other currents become important at particular sites and/or for specific concepts.

A.2.7 Wind turbine

A.2.7.1 Most of the wind turbines available on the market are 3 bladed, pitch controlled, horizontal axis wind turbines (HAWT). Further in the appendix, this kind of wind turbine will be simply referred to as “wind turbine”. No distinction is made in the appendix between direct drive and gearbox technology.

A.2.7.2 Other possible technological choices will include: 2 bladed rotors, vertical axis wind turbines (VAWT), stall regulated machines and downwind rotors. Some specific advices will be given in the appendix, regarding some of these technologies.

A.2.7.3 For a detailed description of wind turbine design, see (DNV and Risø National Laboratory, 2002).

A.2.8 Floaters – general

A.2.8.1 Floaters can be classified in three basic types, based on the way they reach the stability in pitch/roll, see Figure A-2:

— barge, water plane stabilized
— spar, ballast stabilized
— TLP, tendon stabilized.

A.2.8.2 A fourth type is represented by the semisubmersible floater, which obtains pitch restoring from a combination of buoyancy and ballasting. Details about the different floating structures are available from the literature, see (Faltinsen, 1990) and DNV-RP-F205.

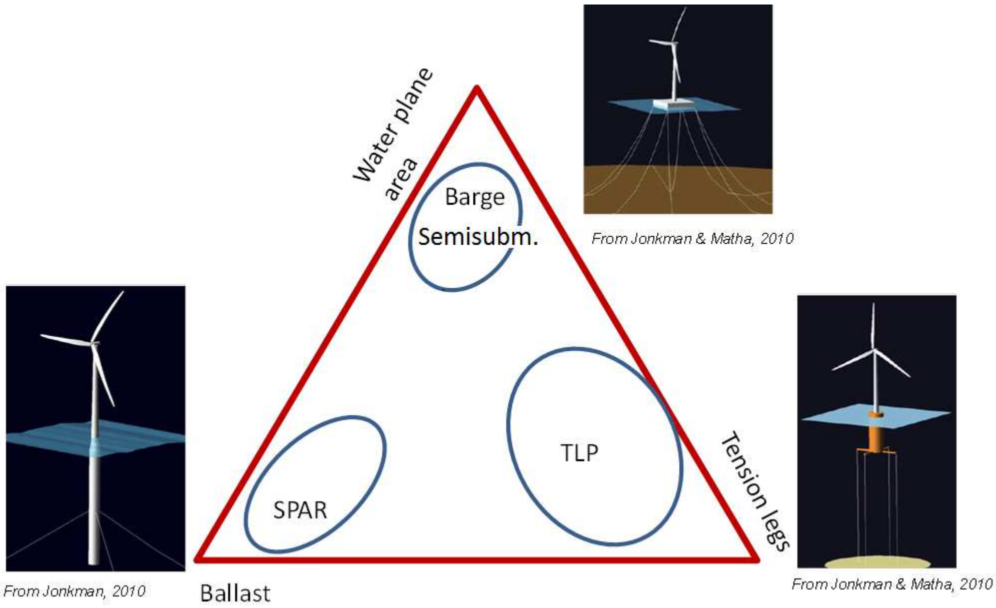


Figure A-2
Classification of the floater, based on the stabilizing principle in pitch

A.2.9 Spars

A.2.9.1 A FWT concept, using a spar platform, has been developed and a prototype "Hywind" has been installed in Norwegian waters (Skaare et al., 2007). Current forces have a large relative importance, and may in many cases be the dominant environmental force contribution. This makes the spar vulnerable to low-frequency Vortex Induced Motions (VIM). Spars usually have small heave motions due to a large natural heave period and in general small wave-frequency motions also in the other degrees of freedom.

A.2.9.2 A few floating wind turbines using a spar platform have been investigated in numerical studies, (Jonkman and Musial, 2010) and (Jonkman and Matha, 2010). In particular in (Jonkman and Matha, 2010), the loads of a floating wind turbine mounted on a spar are compared against the loads experienced by the same wind turbine on land. While most of the loads were almost unchanged, the bending moment at bottom of the wind turbine tower were considerably higher in the floating case, i.e. around 60% larger for the extreme loads and over 100% larger for the equivalent fatigue loads.

A.2.10 Semisubmersibles and barges

A.2.10.1 Semisubmersibles in the oil and gas industry are usually column-stabilized units with relatively small water plane areas. The major part of the buoyancy is typically provided by submerged pontoons that extend beyond the deck-length of the platform. Semisubmersibles may experience motions of significant magnitude, both wave frequent and low frequent.

A.2.10.2 Another buoyancy-stabilized unit is the barge-type floater. This type of floater usually has a large water plane area and correspondingly a high restoring stiffness, especially in heave and pitch but also in roll. This means that the natural periods in these modes of motions are relatively low. In the study by (Jonkman and Matha, 2010), the loads experienced by a wind turbine mounted on a barge are much larger than the loads on the same wind turbine mounted on a spar or on a TLP. These loads seem to be induced by the large displacements of the barge platform.

However, the use of semisubmersible platforms should alleviate these issues, and a wind turbine concept "WindFloat" has successfully been installed off the coast of Portugal, see (Roddier, Cermelli, Aubalt and Weinstein, 2010).

A.2.11 Tension leg platforms – tendon-stabilized units

A.2.11.1 For a Tension Leg Platform (TLP) the vertical motion is governed by the tendon stiffness and not the water plane area. The tendon stiffness also provides the major part of the restoring in heave, roll and pitch, and the stability of a TLP will typically be severely affected if one or more of the tendons lose tension or break.

A.2.11.2 A TLP typically has high natural periods in surge, sway and yaw whereas the natural periods in heave, roll and pitch are low due to the spring stiffness provided by the tendons. The exact resonance periods are given by the length and pretension of the tendons. The resonance periods are also dependent on the flexibility of the wind turbine and the tower.

A.2.11.3 Due to the high restoring in heave, roll and pitch a TLP will typically experience very small wave-frequency oscillations in these modes of motion. Higher order excitation from sum-frequency wave forces may however introduce high-frequency resonant oscillations such as springing and ringing. Such resonant oscillations may give significant contributions to the forces in the tendons, both with respect to fatigue and structural strength. If such resonant behaviour occurs the system is sensitive towards small variations in damping. The damping must hence be properly assessed in order to estimate a realistic response at resonance.

A.2.11.4 When the offset in the horizontal plane becomes large a TLP will experience significant set-down and possibly overturning effects due to the high heave, roll and pitch restoring.

A.2.12 Station keeping system

A.2.12.1 The relevant station keeping systems can be divided into three main groups, namely catenary systems, taut systems and tendon systems.

A.2.12.2 The difference between a catenary system and a taut system is illustrated in Figure A-3.

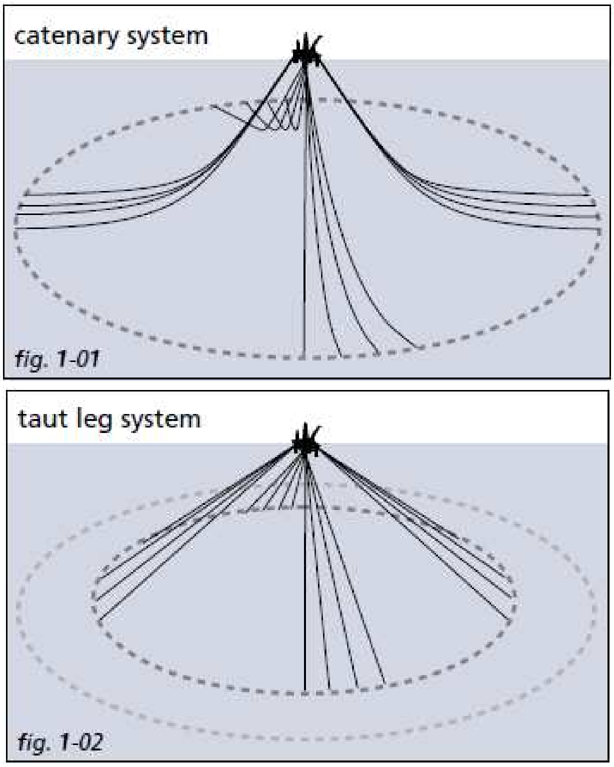


Figure A-3
Difference between catenary and taut mooring systems (from Vryhof, 2012)

A.2.12.3 A tendon system for station keeping is illustrated in Figure A-4.

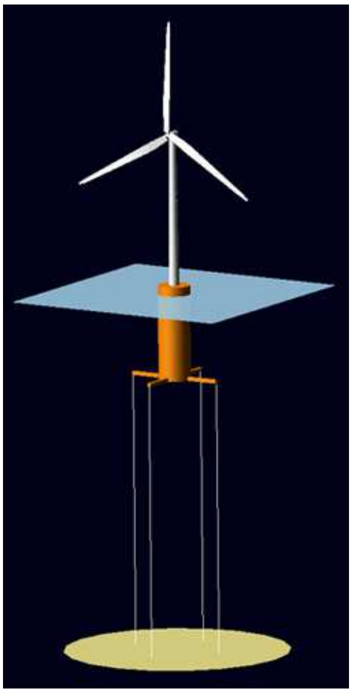


Figure A-4
Example of tendon system for station keeping (from Jonkman and Matha, 2010).

A.2.12.4 In a catenary mooring system, a part of the mooring line towards the anchor lies on the seabed. Most of the restoring force provided by a catenary system is due to the weight of the hanging mooring line, i.e. gravity acting vertically on the line. Catenary systems are commonly composed of steel rope and chain segments, and sometimes clump weights and buoys are used in order to achieve the desired line configuration.

A.2.12.5 In a taut mooring system the mooring line arrives at the anchor with an angle, i.e. no part of the line is resting on the seabed. The mooring line is nearly straight between the fairlead, possible clump weight and the anchor. Therefore the footprint of a taut system in general is smaller than for a catenary system. Most of the restoring force provided by a taut system is due to the elasticity of the mooring lines.

Taut systems often consist of large segments of synthetic ropes.

A.2.12.6 Tendons used in TLPs are usually made of steel tubes rather than elastic ropes. Sometimes synthetic fibre ropes are used. Other than that a tendon system is in many ways similar to a taut system. The steel tubes have little axial flexibility, which is essential in order for the floater to experience only small vertical motions.

A.2.12.7 Anchors transfer forces from the mooring lines or tendons, as applicable, to the seabed soils or rock. Different types of anchors are listed in Sec.9. Some types of anchors lend themselves as feasible for use as shared anchor points for mooring lines from more than one floating unit.

A.3 Recommendations for Models of Environmental Conditions

A.3.1 Wind

A.3.1.1 Turbulent wind shall be modelled according to Table A-2, using one of the spectra presented in DNV-RP-C205. The Mann model for turbulent wind is largely used within the wind energy industry, and it is recommended when the study of the rotor is relevant. Reference is made to IEC 61400-1. Reference is made to Mann (1998).

A.3.1.2 The wind model shall be capable of including:

— turbulence (varying level, resolution and grid variables)
— wind shear
— gust (stochastic and deterministic).

A.3.1.3 Ideally, the time scale should be 10 minutes when aerodynamic loads are included in the simulations; however, for analysis of floating wind turbine units significantly longer simulations are usually necessary.

A.3.1.4 A deterministic representation of the wind can be used for some specific load analysis.

A.3.1.5 In certain cases it may be required to apply different methods when analysing loads due to extreme wind conditions instead of normal wind conditions. As an example it may be relevant to apply measured time series of wind speeds rather than wind speeds generated from generic wind spectra. Some guidance on how to analyse the event of a squall when analysing for use in a mooring system analysis is given in DNV-OS-E301.

A.3.1.6 Caution must be exercised in case of applications to large rotors and long time series simulations, for which it may prove difficult to achieve a sufficiently high resolution of the wind field for the purpose.

A.3.2 Waves

A.3.2.1 The time scale for simulation of waves is usually in the range 3 to 6 hours.

A.3.2.2 In coupled analysis of a floating system, a variety of wave theories can be used, some of which are mentioned in DNV-RP-C205.

A.3.2.3 The most commonly used wave models for coupled analysis are irregular Airy waves. However, the validity of the Airy wave theory may be questioned for shallow water and/or when the waves become steep. In case of very shallow water the waves may significantly change behaviour and become more nonlinear for high crests. For these high crests a nonlinear Stream function or 5th order Stokes wave may be embedded in the irregular Airy wave model to give more accurate wave kinematics around the high crest. To avoid discontinuities a smooth transition between the two wave models must be ensured. For moderate water depth and in deep water 2nd order irregular waves can be used to model high sea states. Nonlinear wave theories shall be used to simulate strongly nonlinear effects like impact from breaking waves.

A.3.2.4 In the process of the floater design analysis, it is recommended that the wave theory applied is adjusted to the actual stage in the design process. For an initial assessment or screening it may be sufficient to apply Airy wave theory for most cases, whereas for detailed design the wave theory should be chosen to be applicable with the governing design conditions. More details on the different wave theories are offered in DNV-RP-C205. For recommendations regarding the selection of the correct theory to a specific project, reference is made to DNV-OS-J101. Numerical tools that can model nonlinear wave theories generally do not apply to calculation of wave loads on large-volume structures pertinent to floating wind turbines.

A.3.3 Currents

A.3.3.1 The currents listed in Table A-2 and [A.2.6.2] have to be modelled according to the specification of the sites. Often the wind and tidal current components are dominant. The different current velocity components can be superimposed by taking the vector sum of each component. It is important to take into account the depth-dependence of the current velocity. If detailed measurements do not exist, this can be done by assigning standard current profiles. Typically the tidal component can be assumed to vary according to a power law, whereas the wind component may be assumed to vary linearly down to some distance between the still water level and the seabed.

A.3.3.2 Currents are in general unsteady, i.e. the velocity, the level of turbulence and the direction vary with time. It is a common assumption in station keeping analysis to assume that the current is stationary (i.e. not varying with time), but varies with depth. For most applications involving global performance of a floater this is a valid assumption.

A.3.3.3 However, for environments where current fluctuations of large magnitude are expected on a time scale that is relevant with respect to the structure’s response, this assumption should be revisited when performing analysis for detailed design. Such considerations usually require current measurements on the actual installation site to be available. In most cases it is still expected that assuming stationary current is applicable also in detailed design, since the global performance of the floater typically is relatively insensitive to local current fluctuations.

A.3.3.4 An accurate modelling of the current conditions is important for determining forces on mooring lines. For structures with large drafts the current forces acting on the hull structure itself may also be significant and in some cases dominant over other environmental loads.

A.3.4 Wave-current interactions

A.3.4.1 For strong currents and relatively steep waves the wave-current interactions should be taken into account when considering air gap and wave impact (slamming) problems, see DNV-RP-C205. If, for example, the current direction is opposite of the wave direction this may tend to steepen the waves.

A.3.4.2 The presence of a current may significantly change the wave drift forces on a floater. This is due to both inviscid and viscous effects, where the former is known as *wave drift damping*. Both of these effects are possible to account for in a computer simulation, reference is made to (Aranha, 1994) and DNV-RP-C205 for how to compute the wave drift damping.

A.4 Recommendations for Dynamic Models

A.4.1 Structural dynamic model

A.4.1.1 When choosing how to model structural responses, it is important to consider the flexibility and the important eigenfrequencies of the structure. In general, relatively simple models can be accepted for structures that have small flexibility and all important structural eigenfrequencies well away from wave and wind excitation frequencies. An example of such a model is a rigid-body model where the structure is restricted from deformation other than rigid body motions, generally in six degrees of freedom.

Guidance note:

For modelling of TLPs it is particularly important to model the flexibility of the tower in a representative manner, as TLPs see significant changes in the natural periods in pitch and roll when the tower is modelled as flexible as compared to rigid. This also applies to compliant tower concepts.

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A.4.1.2 More sophisticated models may represent flexible parts of the structure, e.g. the turbine blades and the turbine tower, with a finite number of nodes. Each of these nodes is free to move in six degrees of freedom (or three if rotations are neglected), and the response of each node is determined from multidimensional stiffness and damping matrices. In order to model the structural response in a realistic way it is thus important that these matrices are properly established so that structural flexibility and damping are duly accounted for.

A.4.1.3 Modal models, finite element models and multibody models may be used for dynamic modelling of the floating wind turbine unit. These three models, and combinations of them, have been widely used in commercially available aeroelastic codes, providing reliable results. Most wind turbine models consist of a combination of these models, for example modal and multibody model or FEM and multibody model.

A.4.1.4 It is recommended that the dynamic structural model be tested and validated, before it is being used for a coupled system analysis to simulate a floating wind turbine. Particular care shall be put also in choosing the correct model, considering the concept to be modelled. It shall be checked that the model can correctly handle the level of deflections, rotations and displacements.

A.4.2 Modal models

A.4.2.1 Modal structural models are computationally efficient, but are not suited to handle effects such as nonlinearities occurring at large deflections.

A.4.3 Finite element modelling

A.4.3.1 A further enhancement is to use a Finite Element (FE) formulation to model the structure. FE methods require higher computational time than modal representations, but may, depending on the applied formulation, take into account effects like large deformations and structural nonlinearities (Ahlström, 2005).

A.4.3.2 When the wind turbine rotor is included in the simulations, the structural model shall include:

— gravity loads on the rotor blades which vary with rotation (most relevant for structural analysis)
— centrifugal and Coriolis forces due to rotation
— gyroscopic forces due to yawing.

A.4.3.3 Structural damping can be added to the model in different ways. However the correct value of the structural damping shall be reproduced and particular accuracy shall be used with the components subjected to vibrations and potential instabilities, such as blades and tower.

A.4.4 Multibody models

A.4.4.1 A multibody formulation divide the model in different bodies, coupled using algebraic equations as constraints. Each of the bodies is modelled with a finite element model. Large rotations and translations are accounted at the coupling interfaces, while small deflections are supposed within the objects.

A.4.5 Aerodynamic model (Aerodynamic loads)

A.4.5.1 Most of the available commercial wind turbines aero-elastic codes use the Blade Element Momentum method (BEM) to calculate the loads on the rotor. The BEM combines the Blade Element and the Momentum theories and it assumes that the rotor can be modelled as an actuator disc, while the flow is axisymmetric.

A.4.5.2 The BEM is particularly suitable for aero-elastic implementations. This is due to the limited amount of the computational time. Thus, even though more complex aerodynamic models are commercially available, the BEM is still widely used in the wind turbine industry for load calculation. This is the case also for floating wind turbines. However, some corrections and improvements must be implemented in the BEM codes when these codes are to be applied to floating wind turbines:

— empirical correction of tip and hub losses
— dynamic inflow wind
— a dynamic stall model
— tower shadow effect.

A.4.5.3 The aerodynamic damping shall be also correctly estimated by the code, including the damping from the non-rotating components such as tower, nacelle and floater.

A.4.5.4 For a description of the state of the art of BEM codes for floating wind turbines, see (UpWind, no date).

A.4.5.5 Generalized Dynamic Wake (GDW) is an aerodynamic model, which implies a different solution method than BEM. The computational efficiency of GDW is similar to that of BEM, only GDW may exhibit convergence problems at large tip speed ratios.

A.4.5.6 Other possible numerical methods that can be used to calculate the aerodynamic loads are:

— a vortex-based method, for example see (Van Garrel, 2003).
— a Navier-Stokes solver (Computational Fluid Dynamics), (NOWITECH, 2010).

Both methods have the disadvantage of relatively long computational times, but can be useful for analysing some aerodynamic aspects in more detail. A comparison of the two methods is available in (Sebastian and Lackner, 2010). These numerical solvers are recommended only for a detailed study of the rotor.

A.4.6 Hydrodynamic model (Hydrodynamic loads)

A.4.6.1 In floating wind turbine analysis, the floater may be modelled as a slender structure (Morison model), a three-dimensional diffracting body or a combination of the two.

A.4.6.2 When the floater is modelled as a slender structure, wave radiation damping may be neglected, and the wave loads on the body can be calculated by means of the semi-empirical Morison's equation which includes a drag term and an inertia term. The use of Morison's equation has several limitations, as summarized in (Jonkman, 2007), (Newman, 1997) and (Faltinsen, 1990):

— wave radiation damping is not considered, which is acceptable only for very rigid structures with small motion
— some of the terms of the added mass matrix are disregarded, which is valid for axial symmetric bodies
— the diffraction problem is simplified according to the G.I. Taylors' long wavelength approximation, which makes the theory valid only for slender bodies, see Figure A-5.

A.4.6.3 An alternative to the Morison's formulation is to model the floater as a three-dimensional diffracting body. Then the wave loads on the body are calculated by means of potential theory, typically through a panel method.

A.4.6.4 Regardless of whether Morison's formulation or potential theory is used to calculate the wave loads, the drag load will not be fully correct. Using Morison's formulation the wave radiation effect is disregarded, and using potential theory the viscous drag is missing. By either method, the wave loads may be solved either linearly or nonlinearly, but none of them accounts properly for viscous effects. Therefore a combination of the methods is sometimes used: Potential theory plus viscous effects from Morison's formulation. Often a slender model is used together with a panel method in order to introduce the effect of viscosity through drag forces on the Morison elements (Jonkman, 2007).

A.4.6.5 The computations can be performed directly in the time domain, or they can provide frequency-domain transfer functions to be used in another time domain simulation code.

A.4.6.6 A guideline for when the various theories are valid is offered in Figure A-5. This figure strictly holds for a fixed circular cylinder with diameter d in waves having wave length L and height H, but is also applicable as guidance for a floating structure with circular-like cross section.

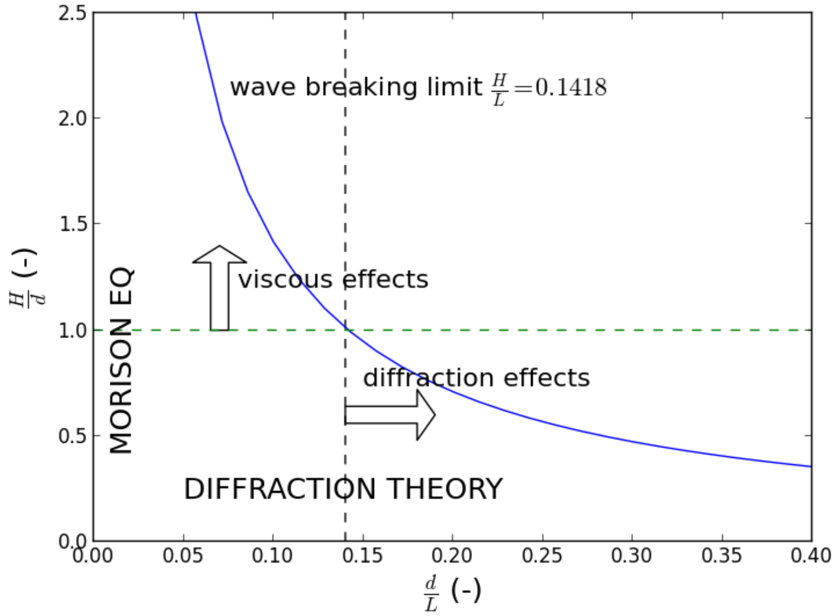

Figure A-5
Regions of validity for different hydrodynamic theories for a fixed circular cylinder with diameter d in waves with wavelength L and height H (based on Kvittem, Bachynski and Moan, 2012).

A.4.6.7 In later years it has become more and more common to apply Computational Fluid Dynamics (CFD) when solving strongly nonlinear fluid-structure interaction problems. Examples of such problems are wave impact (slamming) and ringing loads in steep waves. CFD methods solve the Navier-Stokes equations in the time domain through various numerical schemes, offering a more correct way to compute strongly nonlinear wave loads as well as dealing more properly with viscosity than other methods. CFD is also the most appropriate tool if one is to study VIM numerically.

A.4.6.8 Initial studies may benefit from using simple analysis models such as slender models. Given that the structure is compatible such models may give reasonable estimates during early conceptual phases. For detailed design phases, where for example loads on the tower structure during wave impact in steep waves are needed, it is recommended to use sophisticated methods such as CFD. However, other means to deal with this type of problems exist, applying analytic models for studying effects of wave impact and ringing loads. In either way the results from numerical computations should be duly verified, especially in severe sea states and steep waves. This is commonly done through model testing or comparison with full-scale measurements (if such are available). Hydrodynamic model testing has been performed for decades and is known to give reliable results for most applications when performed properly.

A.5 Models of Subcomponents and Definition of Accuracy Levels

A.5.1 Wind turbine

A.5.1.1 Level I: All the sub components shall be modelled, resembling the correct mass distribution, stiffness and inertia. One of the mentioned structural dynamic models can be used. When the modal model is used, it shall be demonstrated that the system can be considered sufficiently rigid. However the model is considered acceptable for the standard wind turbines, as described in [A.2.7]. The natural frequencies shall be correctly estimated. The aerodynamic model shall be implemented according to the guidelines described in[A.4.5]. If a BEM code is used, the recommended corrections shall be implemented. The aerodynamic damping shall be represented as a function of time (wind), with contributions both from the rotor and from the drag elements (nacelle and tower). The control system shall be implemented. The recommendations in Table A-3 shall be considered, if relevant.

A.5.1.2 Level II: As level I, but some detailed representation of the wind turbine subcomponents can be omitted: the nacelle assembly can be simplified, using concentrated masses and inertial properties. However, the mass distribution and the inertia of the complete system shall resemble the real wind turbine. The control system shall be modelled. Particular care shall be put in use of concentrated masses on rotating elements, depending on how the applied code accounts for the gyroscopic loads.

A.5.1.3 Level III: The tower can be modelled as a spring, reproducing the correct stiffness of the system. Concentrated masses can be used, in order to reproduce the correct global mass distribution (centre of gravity and inertial moments) of the turbine. Gyroscopic effects shall be included if they are relevant. A BEM model should be used for the aerodynamic model but a time series (coupled) could be also used. The loads (the thrust and the moments) can be applied at the hub centre. Aerodynamic damping shall be evaluated separately, considering the dependency on the wind conditions. As an option, the thrust force may be taken from look-up tables which will also provide values for the aerodynamic damping. Note that considering the thrust force only (applied at the hub centre), implies that the yaw moment due to the non-even horizontal distribution of wind speed over the rotor swept area is neglected.

A.5.1.4 Some additional aspects have to be considered for particular rotors that deviate from the reference model which consists of a 3-bladed, pitch-controlled HAWT, see [A.2.7]. The most relevant aspects in this context are highlighted in Table A-3.

Table A-3 Additional aspects for evaluation of the aerodynamics loads

Deviation from the reference model	*Need for investigation*	*Recommended analysis*	*References and notes*
VAWT	The induction model needs to be changed	Use of double disc models or more complex (Cylinder actuator model)	(Sutherland, Berg and Ashwill, 2012), (Madsen, Paulsen and Vita, 2012)
	Wake effects are more important	Vortex model (or circulation models) for rotor evaluation	(Ferreira, 2009)
Downwind rotors	The rotor is in the shadow of the tower and the nacelle	Accurate model of the shadow effect of the tower and nacelle	
Multiple rotors (HAWT)	Wake effects should be included	Wake models to study the wakes interaction	
Multiple rotors (VAWT)	Wake are asymmetric	Wake models to study the wakes interaction	

A.5.2 Floater

A.5.2.1 Level I: The hydrodynamic model shall be according to the guidelines given above, including diffraction and radiation contributions. CFD shall be used for solving nonlinear problems, which are relevant for the concept, see [A.6.3]. The correct mass distribution and stiffness properties shall be reproduced. For simulations in the time domain, the damping and added mass of the floater shall be determined for the range of frequencies of the simulation. Control of the floater shall be included, if there is any. The recommendations in Table A-4 shall be considered if relevant for the concept.

A.5.2.2 Level II: As level I, but Morison's equation can be used when suitable for the concept. Corrections shall be implemented as described in [A.4.5]. Some simplifications can be introduced depending on the concept. The inertia and drag coefficients can be considered constant over time, provided that the variation is limited in the frequency range of the simulation.

A.5.2.3 Level III: The model of the floater can be simplified as a spring (reproducing the correct mass properties, stiffness and periods). The damping shall be evaluated and added to the system. Loads can be added with a time series or with a look-up table (given the displacements of the system).

A.5.2.4 In addition to the recommendations in [A.5.2.1] to [A.5.2.3], some additional aspects need to be considered, depending on the particular floater to be analysed, see Table A-4.

Table A-4 Additional aspects to consider in the analysis of the floater

Floater type	*Aspect to be included in the analysis*	*Accuracy level requiring the analysis*	*References and notes*
Spar	VIM and cross-flow oscillations.		See [A.6.5]
	Increased added mass and drag due to strakes (if such are fitted).		
	Ringing loads on column(s) in steep waves.		
	Coupled motions in heave and roll/ pitch, Mathieu-type instability.		
	Viscous hull damping, mainly due to vortex shedding.		
	Roll-yaw coupling		

Table A-4 Additional aspects to consider in the analysis of the floater (Continued)

Floater type	*Aspect to be included in the analysis*	*Accuracy level requiring the analysis*	*References and notes*
Semisubmersible	Air gap and associated slamming loads (if negative).		
	Viscous drag on columns, pontoons and bracing.		
	Added-mass forces on bracing.		
	Coupled motions in heave and roll/pitch.		
TLP	Sum-frequency effects		
	Ringing loads on column(s) in steep waves.		
	Damping in heave, roll and pitch.		

A.5.3 Station keeping system

A.5.3.1 In terms of modelling mooring lines in a numerical analysis two alternatives exist, commonly denoted *quasi-static* or *dynamic* modelling. In brief one may say that the mooring system in a quasi-static analysis is modelled as nonlinear springs and hence does not include mass and drag forces acting along the length of the mooring lines. In a dynamic analysis the mooring lines are modelled as slender elements so that mass and drag forces acting along the length of the line are included.

A.5.3.2 The need to use a dynamic mooring line model in the analysis traditionally increases with the water depth, since the mooring line dynamics become more important the longer the span of the line. Also, the mooring line damping is larger in deep water. The term "*deep water*" must here be seen both relative to the dimension of the floating structure and its mooring footprint and relative to the relevant wave condition. The deep-water limit for a floating wind turbine will typically be shallower than for an oil and gas installation such as an FPSO or a semisubmersible.

A.5.3.3 In a mooring analysis it will most often be sufficient to consider the anchor points as being prescribed and fixed. In this case no further geotechnical investigations than those carried out for the prescribed anchor points are required. For anchors used as shared anchor points for mooring lines from two or more floating units, it is important to consider the loading from all attached mooring lines.

A.5.3.4 Depending on the type of mooring lines to model, some specific aspects have to be considered.

A.5.4 Taut mooring lines

A.5.4.1 For a taut mooring system or a TLP the soil may contribute with significant damping. For these types of station keeping systems the soil damping should be assessed with an appropriate level of detail. If not a conservative estimate should be applied.

A.5.4.2 Since the mooring lines in a taut system usually display smaller transverse motions through the fluid compared to in a catenary system, the drag forces on the mooring lines are smaller and the dynamic effects on the floater moderate.

A.5.4.3 Taut systems often consist of large segments of synthetic ropes. It should be noted that such ropes exhibit more complex force-elongation characteristics than steel ropes (e.g. hysteresis). This should be taken into consideration in an analysis

A.5.5 Catenary mooring lines

A.5.5.1 Catenary systems can usually be represented by standard catenary formulations that relate submerged weight of suspended line, horizontal mooring load, line tension and line slope at the fairlead.

A.5.5.2 Since the transverse motion of the mooring lines in a catenary system may be significant, drag forces on the mooring lines may also be significant. This is commonly referred to as anchor-line damping.

A.5.5.3 Level I: Dynamic model. The model shall reproduce the real dynamics of the mooring lines. FEM models can be used. The buoyancy and the drag of the lines shall be included.

A.5.5.4 Level II: Quasi static model.

A.5.5.5 Level III: The loads can be calculated separately by means of dedicated software and a look up table can be used.

A.5.6 Tendons

A.5.6.1 The station keeping forces from a tendon system are essentially governed by the length and pretension of the tendons.

A.5.7 Control system models

A.5.7.1 The wind turbines need a control strategy in order to regulate the power output, limit the loads and avoid overspeeding at high wind speeds. There are three main principles to control the power of a wind turbine:

— Pitch control: at low wind speeds, the pitch angle of the blades is regulated in order to maximize the power production. Above the rated wind speed, the blades are pitched towards feather to decrease the lift and keep constant power. A pitch controlled wind turbine maintains constant rated power at high speed, as in Figure A-6.
— Active stall: The turbine is controlled as with the pitch control up to the rated wind speed. Above the rated wind speed, the blades are pitched in order to cause aerodynamic stall of the blades. The power output will decrease beyond the rated wind speed.
— Passive stall: The pitch angle of the blades is fixed and cannot be changed. The turbine reaches aerodynamic stall at high wind speeds, because of the aerodynamic design of the blades (Ahlström, 2005). The stall reduces the power at high wind speeds.

A more detailed description of different wind turbines control systems is available in (DNV and Risø National Laboratory, 2002).

A.5.7.2 The vast majority of the wind turbines on the market at the time of this standard are pitch controlled. However active stall could be relevant for floating wind turbine in the future (Larsen and Hanson, 2007), while passive stall is commonly used to control VAWT.

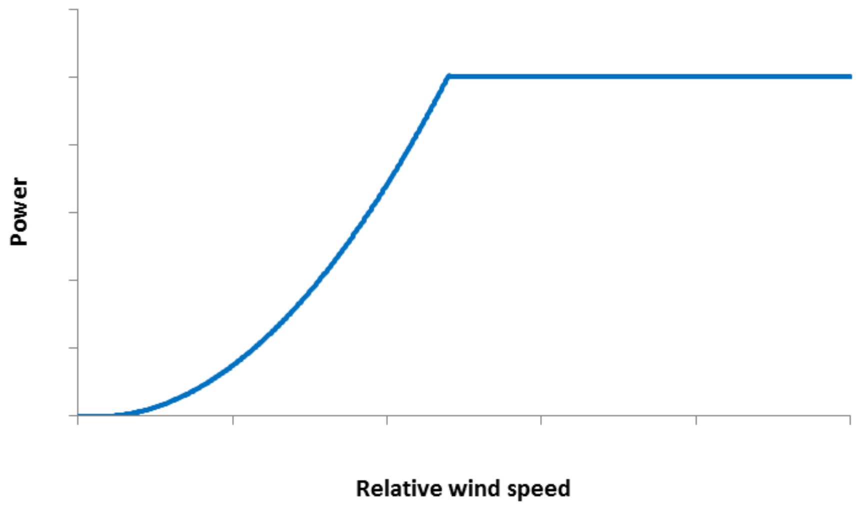

Figure A-6
Illustration of a typical power-generation curve for a pitch-controlled wind turbine

A.5.7.3 Regardless of the type of wind turbine, a correct implementation of the control system is fundamental for:

— a correct estimation of the performance
— a correct evaluation of the loads
— the verification of the dynamic response of the full coupled system.

In order to achieve this, it is important that the model of the control system will reproduce the following quantities in a realistic way:

— the main parameters of the control (time of the activation, activation delay, speed of the pitch, maximum angle of pitch)
— the frequency of the control system, i.e. the frequency with which the "operation loop" of the control system is executed, where one loop in the case of a pitch controller consists of collecting the generator rpm as input and returning the correct pitch angle of the blades as output
— the correct aerodynamics characteristics of the airfoils (obtained by experimental test)
— the effects of the control system on the rest of the wind turbine system, such as the generator control and the safety system.

A.5.7.4 Application of a typical control system, meant for onshore wind turbines, to a floating wind turbine can create unwanted dynamic effects, as shown by (Larsen and Hanson, 2007) and (Skaare et al., 2007) for a wind turbine on a spar. Therefore floating wind turbines supported by spars need a control system with a frequency which is lower than the pitch/roll frequency of the floating unit, such that the unwanted effects of the control system on the aerodynamic damping can be avoided and the floater motions can be kept at a limited level. Note that the lower frequency of the controller leads to a less effective regulation of power.

A.5.7.5 In line with [A.5.7.4], a control system for a barge-supported floating wind turbine, presented by (Lackner, 2009), uses an individual blade pitch controller (IPC) in order to reduce the floater motions and the fatigue loads on the wind turbine blades.

A.5.7.6 The control system can thus strongly affect the dynamic stability of a floating wind turbine. The controller can actively be used to maintain the stability of the floating system, for instance by controlling the overall magnitude of thrust on the wind turbine rotor. This implies that the control system is a fundamental component in a coupled aerodynamic-hydrodynamic-servo-elastic system and must be properly modelled when this system is modelled.

A.5.7.7 The control system of a turbine is not only characterized by its frequency, but also by specific procedures to accomplish the wind turbine safety strategy. The model of the control system shall therefore be able to reproduce the activation of the safety system and the actuation of the safety procedure for stopping the wind turbine. This implies that the model of the control system shall cover

— the conditions at which the safety procedure is activated (for example maximum rpm of the rotor, maximum pitch/roll of the floater, failure of the mooring line)
— the way in which the safety procedure is activated (for example speed of blade pitching, application of mechanical torque).

A.5.7.8 If the wind turbine or any subcomponent has any additional control system which will influence the dynamic response of the floater, also this system must be properly modelled.

A.5.8 Other external forcing

A.5.8.1 In some areas of the world it may be relevant to include loads from other sources such as drifting ice. The approach adopted to take into account such loads and corresponding responses must be chosen with basis in the physical problem studied and the level of detail that is required. A simple example of such procedure may be to apply the time history (magnitude and direction) of an external force (or moment) in a fixed point of attack.

A.5.9 Recommendations for the coupling interfaces

A.5.9.1 It is already established that the coupling between the force on the wind turbine and the motion of the floater is crucial to account for, and this applies for all types of concepts. Much of the discussion is based on DNV-RP-F205 along with DNV-OS-J101 and DNV-RP-C205.

A.5.9.2 The interfacing scheme applied by (Jonkman, 2007) to obtain a coupled aero-hydro-servo-elastic analysis model is illustrated in Figure A-7. The term *aero-hydro-servo-elastic* coupling implies a coupling between the aerodynamic loads and responses, the hydrodynamic loads and responses, the control system and the deformation response of the structure due to elasticity. No coupling is usually modelled between the environmental conditions, i.e. the sea state is not related to the inflow wind.

A.5.9.3 A description of different coupling strategies is given in (Roddier, Cermelli, Aubalt and Weinstein, 2010).

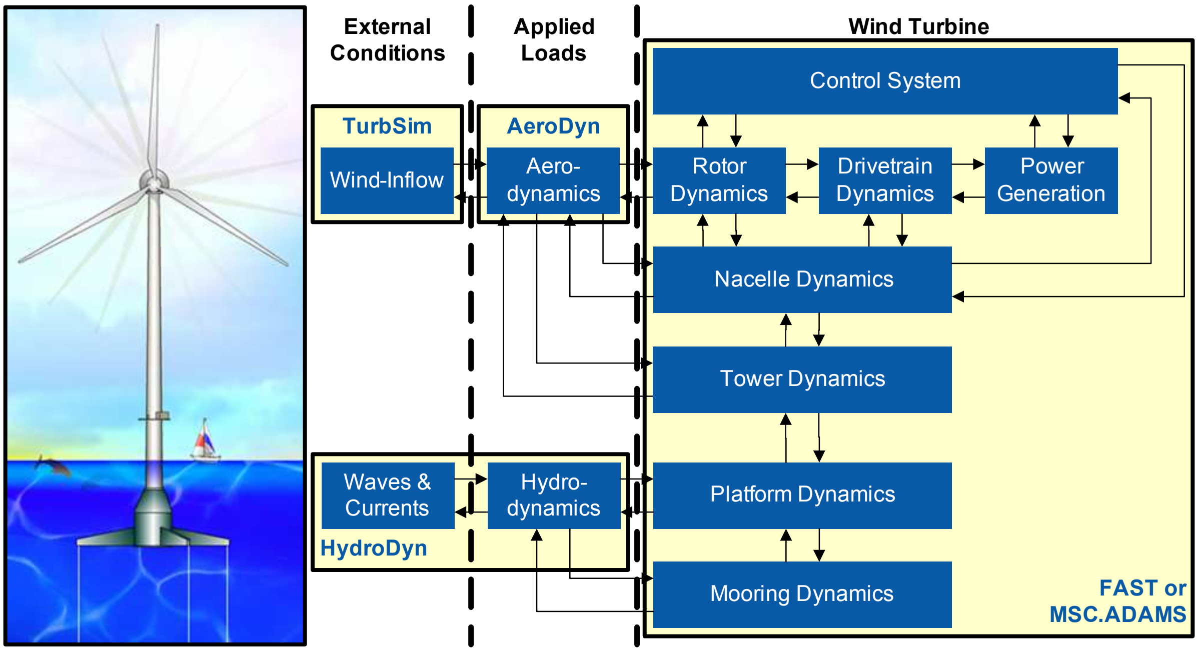

Figure A-7
Example of interfacing modules applied to achieve aero-hydro-servo-elastic coupled analysis (from Jonkman, 2013)

A.5.9.4 It is noted that in a coupled analysis, all forcing and response equations are solved simultaneously (i.e. at each time step). Various time stepping methods exist to evolve the solution in the time domain. These differ with respect to efficiency, accuracy and stability. Common methods include time integration schemes like Euler, Runge-Kutta and Newton-Raphson. Different time stepping methods may be relevant for the various sub-problems in the analysis. Differential-Algebraic-Equation (DAE) solvers are often needed in coupled problems.

A.5.9.5 For a floating wind turbine there is a strong coupling between the floater motions and the turbine forces. This coupling can briefly be outlined as follows:

— The motion of the floater will influence the aerodynamic inflow conditions to the turbine.
— The forces acting on the turbine will influence the motion of the floater mainly through excitation and damping forces. The natural periods of the floater motion may be altered by the wind turbine.

A.5.9.6 The aerodynamics of the wind turbine can contribute to the floater stability with a positive or negative damping, depending on the external conditions and the control system. A description of the effects of the aerodynamic loads on the platform motion is given in (Nielsen, Hanson and Skaare, 2006) and (Nielsen, F. G., no date) for a spar and (Lackner, 2009) for a barge. For some types of installations there may also exist an important coupling between the station keeping system and the motions of the floater. This coupling can briefly be outlined as follows:

— The wave frequency motion of the floater will influence the forces in the mooring system
— The motion of the station keeping system will exert forces on the floater, generally both in terms of restoring forces, damping forces and possible excitation forces.

A.6 Analysis Experience from the Offshore Industry

A.6.1 General

A.6.1.1 Some advices are given for the application of the models described in [A.5.1] to [A.5.9].

A.6.1.2 Although the concept of floating wind turbines may seem rather different from floating oil and gas installations, there are many similarities. For instance the floater and mooring concepts applied for floating wind turbines and oil installations are similar, although the physical scale obviously is different. Based on this understanding, one may utilize experience gained from the oil and gas industry in the analysis of floating wind turbines with respect to several topics including:

— coupling between floater motions and mooring system
— how to analyse nonlinear wave and response phenomena

— how to analyse extreme wave loads (e.g. slamming, ringing)
— how to analyse extreme wind loads
— how to analyse current-induced responses (VIM)
— shallow-water effects
— sloshing
— wave-current interactions.

A.6.2 Coupling between floater motions and station keeping system

A.6.2.1 It is well established in the oil and gas industry that the necessity of performing a coupled analysis, in which floater motions, mooring lines and their interaction are taken into account simultaneously, increases with increasing water depth. This is both due to dynamic amplification of the mooring line forces and due to increased mooring line damping.

A.6.2.2 For typical floating offshore oil and gas installations it is often said that it becomes necessary to perform a coupled mooring analysis for water depths greater than 150 m. This limit is based on experience from the oil and gas industry and implicitly takes the dimensions of oil and gas floaters and their mooring systems into consideration. For a floating wind turbine, with considerably smaller dimensions, it is necessary to perform coupled analysis also in shallower waters.

A.6.2.3 When establishing the necessity to perform coupled analysis the natural periods of the floater and the mooring system have to be taken into account. If, for example, the resonant wave-frequency pitch period is near the (modal) natural period of the mooring lines, it is to expect that coupled analysis will reveal significant dynamic amplification in the mooring line forces. The coupling between the wind turbine action and the floater motions further complicates these considerations.

A.6.3 Nonlinear wave loads and responses

A.6.3.1 When nonlinear effects are important, time domain analysis is preferred. If the problem is analysed in the frequency domain the nonlinear effects may be linearized, but this is not always a suitable approach. In this context, nonlinear effects are effects that are quadratic or otherwise higher-order in either the wave amplitude or the floater motion amplitude.

A.6.3.2 Nonlinear wave load effects on a floater include:

— nonlinear restoring forces due to varying water plane area
— nonlinear damping due to viscous effects
— nonlinear inviscid wave load contributions from structural parts above still water level and effects due to large flare
— sum and difference frequency loads.

A.6.3.3 Nonlinear responses include:

— nonlinear roll motion
— nonlinear coupling between different modes of motion
— (parametric) Mathieu instability
— slow-drift motion
— springing and ringing.

A.6.3.4 Some nonlinearities can be accounted for in a time domain simulation with linear wave force transfer functions by including Morison elements, whereas in other cases the hydrodynamics needs to be solved directly in the time domain.

A.6.4 Extreme wave loads

A.6.4.1 Extreme wave loads due to for example slamming and ringing must be analysed in the time domain. When very refined analysis methods are required, such as CFD, the associated analysis cost may be large. In such cases it is standard practice to only analyse critical events, e.g. analyse a specified wave cycle that is found to generate critical slamming loads. The critical events can typically be indicated by simpler models where full short-term sea states can be analysed at a reasonable computational cost.

A.6.4.2 Several approaches exist for estimating wave loads due to slamming/wave impact, see DNV-RP-C205. Wave impact loads may also be estimated by means of classical methods due to Von Karman or Wagner (Faltinsen, 1990) or by applying potential theory through a Boundary Element Method. However, due to increased computational capacity it becomes more and more common to analyse wave impact problems with CFD methods. Depending on the method this offers the most detailed modelling of the physical effects involved, and it also in general allows for modelling of the steep waves that may be relevant. However, as for any advanced method, the results must be properly verified. This can be done e.g. by comparison with model tests.

A.6.4.3 The so-called ringing loads, which are higher-order wave loads relevant for survival conditions, may also be analysed by means of CFD. However, alternative methods that are less computationally demanding exist. One of these is the so-called FNV (Faltinsen, Newman and Vinje) method, see (Faltinsen, Newman and Vinje, 1995).

A.6.4.4 For water entry loads it may be necessary to consider hydroelastic effects. This may for example be relevant for stiffened plates. Hydroelasticity implies that the structural deformation following an impact influences the hydrodynamic pressure. However, it is generally conservative to neglect hydroelastic effects, see DNV-RP-C205, item 8.7.3.

A.6.4.5 Experience shows that analysing loads due to extreme waves is difficult even when the most sophisticated methods are applied. This is mainly due to the extremely nonlinear nature of such loads. Proper verification is thus needed when analysing such loads analytically or numerically, and often it will be required to carry out model tests.

A.6.4.6 In certain areas of the world one may experience a significant amount of crossed seas, e.g. large swell waves coming from one direction and large wind-generated waves from another direction. This should be taken duly into account in a numerical analysis, as it may significantly change the response of the floater.

A.6.5 Current-induced responses

A.6.5.1 Under certain condition VIM response may be induced by the current. This has been observed to be a problem for DDFs such as spar buoys and may lead to a significant increase in the mooring line forces.

A.6.5.2 Also for mooring lines the possibility of VIM should be assessed. In this connection VIM is especially relevant in terms of fatigue capacity and not so much for ultimate strength.

A.6.5.3 Studies have shown that predicting VIM response is an extremely difficult task with the state-of-the-art tools presently available, and careful model testing is likely to be required.

A.6.6 Shallow-water effects

A.6.6.1 The water depth compared to the relevant on-site wave lengths defines the validity of different wave theories. For very shallow waters, the Airy and Stokes wave theories are insufficient, and one may have to use other wave theories such as the Stream function wave theory. As the water depth decreases the probability of breaking waves, during which the wave kinematics are dramatically changed, also increases.

A.6.6.2 The wave loads and response characteristics (e.g. added mass) may change significantly in shallow waters. In this case the shallowness of the waves should be determined both compared to the wave length and the draft of the analysed floater. One example is that the off-diagonal terms in the Quadratic Transfer Function (QTF) wave load matrices become increasingly important. Including the off-diagonal terms in the QTF significantly increases the computation cost of the analysis.

A.6.7 Sloshing

A.6.7.1 Violent fluid motion in internal tanks is denoted sloshing. Sloshing may represent a hazard to the fatigue life of internal structural members, and may also alter the motion characteristics of the floater. For floating wind turbines that use ballast tanks with internal free surfaces to provide stability, the probability of sloshing and sloshing-related problems should thus be assessed. A comprehensive overview of sloshing aspects is offered by (Faltinsen and Timokha, 2009) where sloshing is discussed on a higher level and where details on how sloshing can be analysed numerically are provided. Analytical formulas presented in this reference can be used to assess the natural sloshing frequencies in a tank, and this information can be used to determine if a more detailed sloshing study is necessary.

A.6.7.2 It is also to be noted that some hydrodynamic codes are capable of computing the coupled floater motion and internal fluid motion in a linear fashion. Results from such calculations may be used to study the influence of sloshing in a larger perspective, but fails to predict the sloshing response accurately at the natural sloshing periods. This is due to the fact that sloshing in reality is a strongly nonlinear phenomenon. It is noted that sloshing in tanks excited by vertical motions is a nonlinear phenomenon that will not be properly reflected by linear analysis.

A.7 Use of Model Tests

A.7.1 General

A.7.1.1 Both wind turbines and floaters have been widely tested in dedicated experimental tests in wind tunnels and ocean basins, respectively. The challenge of model tests of floating wind turbines consists in coupling the model of the wind turbine and the model of the floater, capturing all the necessary dynamics.

A.7.1.2 When performing model tests, especially for novel concepts, the strategy should always be to start out with a simple case and then add complexity in a step-by-step procedure. It is common for a floating offshore structure to first do tests in current alone and then in waves alone before doing tests with both current and waves present at the same time. When using model test results to calibrate and/or verify a numerical model such an approach is crucial, because it enables the analyst to control the different environmental load contributions. It is also a fact that there is an interaction between waves and current (wave-current interaction). If a successful comparison is performed between the model test and numerical model in current alone and waves alone but not when current and waves are present at the same time, this could for example indicate that the wave-current interaction in the numerical model should be subject to investigation.

It is also possible to carry out dedicated experiments on the separate subcomponents, before testing the full coupled model.

A.7.1.3 Wave testing of floating structures has been going on for decades and utilizes well-established methods. The limitations in this respect are more related to the capabilities of different wave basins than to the applied procedures. The sophistication of the wave maker system in different model basins varies, and there are large differences in current-generation systems with respect to generating controlled current conditions. These issues related to modelling of the physical environment need to be addressed when selecting the model test facility to be used.

A.7.1.4 For offshore floaters the wind forces are usually applied through constant drag coefficients found from wind tunnel testing or from computer simulations. Wind turbines, on the other hand, introduce loads that in reality are time-varying. It is however challenging to control the wind conditions in a model basin on a detailed level (wind profile, level of turbulence), and similarly as for current generation systems the quality of the wind maker system also varies significantly from one basin to another. This means that it may be a more robust procedure in a model test to model the wind turbine as an equivalent plate or similar (as was done in (Roddier, Cermelli, Aubalt and Weinstein, 2010)). Such tests are possible to reproduce in a coupled computer simulation without too many obstacles, and hence provide a good means to calibrate the software and to verify that the coupling is well-handled on an overall level. When doing calibration and/or verification of numerical analysis tools it is crucial that one has good control of both forcing loads and response characteristics, and this is easier to achieve if the tests are kept within current state-of-the-art capabilities.

A.7.1.5 Further enhancements in modelling the wind turbine, i.e. including the rotating blades and possibly also the structural flexibility, seem very challenging in a basin model test. This is associated with the wind modelling as well as with the general complexity of the turbine model. It thus seems as if one will not be able to use such detailed tests to a similar extent when doing calibration and verification of numerical software due to the increased level of uncertainty both with respect to forcing and response. The tests can however give valuable insight in the actual physical behaviour of the system. In such tests it may also be considered to model the control system of the wind turbine.

A.7.1.6 When doing wave model testing of floating structures the station keeping system has to be modelled in the basin. In advanced tests one may model the exact mooring system relatively correctly in model scale, but often one relies on a more simplified system called a *soft mooring system*. In such system the floater is attached to several springs that are mounted to represent the actual mooring system. When designing the soft mooring system care must be taken when determining the attachment points on the floater as well as the stiffness of the mooring lines. The soft mooring system should be attached so that the attack point of the mooring forces is as close as possible to that of the real mooring system, and the spring stiffness should be chosen so that the natural periods of the floater are realistic.

A.7.1.7 When modelling the station keeping system for a TLP, special care must be taken when the effect of tether slack is evaluated. It is also experienced that the vertical high-frequency (ringing/springing) response of a TLP is very sensitive towards damping, so that it should be aimed to achieve a realistic system damping when model testing a TLP supported wind turbine concept.

A.8 Assessment of Results

A.8.1 General

A.8.1.1 Periods, damping, stiffness and mass distribution should be evaluated in a first stage.

A.8.1.2 Loads should be evaluated with a simulation of the full system. Some specific loads can be calculated in dedicated calculations (for example second order loads on floater), when it is provided that they can be de-coupled from the model (or applied as an external load).

A.8.1.3 When a commercial wind turbine is used, a comparison against a land based version is recommended, as in (Jonkman and Matha, 2010) and (Jonkman and Musial, 2010)

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APPENDIX B REGIONAL ENVIRONMENTAL DATA

Table B-1 Wave data for various regions and various return periods

		Significant wave height H_S (m) in 3-hr stationary sea state			
Return period (yrs)		1	50	100	1000
Notation		H_{S1}	H_{S50}	H_{S100}	H_{S1000}
Region	Southern North Sea (Egmond aan Zee)	6.3	8.1	8.4	9.5
	Danish North Sea	8.1	10.8	11.3	12.8
	Northern North Sea	11.0	14.2	14.9	16.5
	Norwegian Sea	11.6	15.4	16.0	18.0
	Barents Sea	10.0	13.6	14.5	16.3
	Sea of Japan	10.5	14.6	15.3	17.7
	North Pacific (southeast of Japan)	10.7	13.9	14.5	16.3
	East China Sea	10.2	12.9	13.4	14.9
	Yellow Sea	9.4	12.5	13.1	14.9
	Bay of Biscay	12.5	16.0	16.7	18.8
	Southern Bay of Biscay (Estaca de Bares)	9.8	12.4	12.9	14.5
	Western Mediterranean Sea (Tarragona)	5.4	6.5	6.9	7.6
	Northeast Pacific (west of Oregon)	9.0	16.1	17.4	21.5

Table B-2 Zero-upcrossing period conditioned on significant wave height

Region	*Coefficients in expressions for mean and standard deviation*			
	a_1	a_2	b_1	b_2
Northern North Sea	0.935	0.222	0.1386	–0.0208
Norwegian Sea	1.125	0.150	0.0978	–0.0074
Barents Sea	0.974	0.205	0.1263	–0.0201
Sea of Japan	1.310	0.121	0.4006	–0.2123
North Pacific (southeast of Japan)	0.976	0.197	0.1288	–0.0184
East China Sea	1.044	0.161	0.1166	–0.0158
Yellow Sea	0.841	0.241	0.1977	–0.0498
Bay of Biscay	1.197	0.135	0.0954	–0.0083

Table B-3 Wind data for various regions and various return periods

		10-minute mean wind speed U_{10} (m/s) , 10 m above SWL			
Return period (yrs)		1	50	100	1000
Notation		$U_{10,1}$	$U_{10,50}$	$U_{10,100}$	$U_{10,1000}$
Region	Northern North Sea	36	43	45	48
	Norwegian Sea	32	39	40	44
	Sea of Japan	29	34	35	37
	North Pacific (southeast of Japan)	36	43	44	48
	East China Sea	36	39	41	45
	Northeast Pacific (west of Oregon)	20	31	33	39