

Guide for Building and Classing

Bottom-Founded Offshore Wind Turbines



July 2020



GUIDE FOR BUILDING AND CLASSING

BOTTOM-FOUNDED OFFSHORE WIND TURBINES
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American Bureau of Shipping
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Foreword (1 July 2020)

This Guide provides criteria for the design, construction, installation and survey of Bottom-Founded Offshore Wind Turbines, which possess permanently sited substructures and foundations of offshore wind turbines attached on and supported by the sea floor.

The criteria applicable to floating offshore wind turbines are provided in the *ABS Guide for Building and Classing Floating Offshore Wind Turbines (FOWT Guide)*.

Requirements for ancillary offshore wind farm installations such as meteorological measuring towers, accommodation units, and transformer platforms are not addressed in this Guide. For the requirements for such bottom-founded installations, refer to the *ABS Rules for Building and Classing Offshore Installations*.

This revision incorporates the following changes:

- “Installations” or “Installation” as in context “Bottom-founded Offshore Wind Turbine Installations” or “Bottom-founded Offshore Wind Turbine Installation” deleted.
- 1-1/5.1 – Additional requirements of optional class notation “RNA” for the Rotor-Nacelle Assembly (RNA) and the tower added.
- 1-1/5.3 – New mandatory class notation for “Design Fatigue Life” added.
- 1-1/9.7 – Information for wind turbine control system and power cable system in information memorandum added.
- Section 3-2 – Storm wind models modified to incorporate the recent updates in API RP 2MET.
- 3-2/13 – A new Subsection “Survival Wind Conditions” to align with the *FOWT Guide* and IEC 61400-3-1 added.
- 3-3/3 – For Normal Sea State (NSS), wave model simplified to align with IEC 61400-3-1.
- 3-3/5 – For Severe Sea State (SSS), wave model simplified to align with IEC 61400-3-1.
- 3-3/7 – For Extreme Sea State (ESS), wave model simplified to align with IEC 61400-3-1.
- 3-3/9 – A new Subsection “Survival Sea State (SurSS)” to align with the *FOWT Guide* and IEC 61400-3-1 added.
- 3-4/7 – A new Subsection “Survival Current Model (SurCM)” to align with the *FOWT Guide* and IEC 61400-3-1 added.
- 3-5/7 – A new Subsection “Survival Water Level Range (SurWLR)” to align with the *FOWT Guide* and IEC 61400-3-1 added.
- 4-2/3 – Design Load Cases (DLCs) modified to incorporate the recent updates in the IEC 61400-3-1.
- 4-2/Table 1 – New design load case (new DLC 2.5) added to align with IEC 61400-3-1.
- 4-2/5 – A new Subsection “Definition of Survival Load Cases (SLCs)” added to align with the *FOWT Guide* and IEC 61400-3-1.
- Section 5-5 – “Marine Operations” moved to Chapter 5 to align with *ABS Rules for Building and Classing Offshore Installations*.
- Chapter 6 – A new Chapter “Equipment and Systems” added to cover other equipment and systems including fire fighting systems and equipment, safety outfitting, and identification marks. These requirements are adapted from the *FOWT Guide* and are in line with IEC 61400-3-1.
- Chapter 7 – The “Surveys During Construction, Installation, and Commissioning” and “Surveys After Construction” moved to the new Chapter 7 “Surveys” to align with *ABS Rules for Building and Classing Offshore Installations*.

- 7-1/7 – Modified requirements for installation and commissioning surveys for foundation, substructure, RNA and tower. These requirements are in line with the *FOWT Guide* and IEC 61400-3-1.
- 7-2/1.5 – A new Subsection on “In Service Inspection Plan” added.
- Appendix A1-1 – Editorial changes made for the wind models to align with API RP 2MET.
- Appendix A2-1– Editorial changes made for the tropical cyclone wind model.
- Appendix A4-1/1 – Abbreviation for “NPD” added.
- Appendix A4-1/3 – References for new revisions of *ABS Guides* and *Rules*, and API and IEC standards added.

This Guide becomes effective on the first day of the month of publication.

Users are advised to check periodically on the ABS website www.eagle.org to verify that this version of this Guide is the most current.

We welcome your feedback. Comments or suggestions can be sent electronically by email to rsd@eagle.org.



GUIDE FOR BUILDING AND CLASSING

BOTTOM-FOUNDED OFFSHORE WIND TURBINES

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CHAPTER 1

Conditions of Classification

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CHAPTER 1 Conditions of Classification

SECTION 1 Scope and Conditions of Classification

1 Classification (1 July 2020)

The general requirements for conditions of classification are contained in the *ABS Rules for Conditions of Classification – Offshore Units and Structures (Part 1)*.

Additional requirements specific to Bottom-founded Offshore Wind Turbines are provided in the following Subsections.

3 Classification Symbols and Notations (1 July 2020)

The following class notations are designated for the Bottom-founded Offshore Wind Turbine as defined in 1-2/3.3. Additionally, at the request of the Owner, the Rotor-Nacelle Assembly (RNA) and tower may be included in the classification. An optional notation for the RNA and the tower will be added to ABS Record upon the satisfactory compliance with the requirements given in 1-1/5.1.

Where the RNA and tower are not included in the classification, the RNA and tower installed on the ABS classed Bottom-founded Offshore Wind Turbine are required to have a type certificate in accordance with IECRE OD-501 or other recognized standards. The type certificate will be reviewed by ABS solely to confirm it is current and verify that the information of the installed RNA and tower is consistent with the design information, criteria and limitations considered in the classification of the Bottom-founded Offshore Wind Turbine. ABS will not review or be responsible for the accuracy of the type certificate.

Where the tower and its connection to the RNA are not covered by the wind turbine type certificate and the optional class notation is not requested, the tower and its connection to the RNA are to be included in the scope of classification.

Where interface structures of the tower connecting to the substructure are not covered by the wind turbine type certificate and the optional class notation is not requested, these interface structures are to be included in the scope of classification.

Use of the RNA which does not have a type certificate or the wind turbine having a non-horizontal-axis configuration are subject to special consideration by ABS. Design documents are to be submitted for review. Scope of the design review is to be defined on a case-by-case basis.

In this Guide, the Bottom-founded Offshore Wind Turbine is considered as an unmanned structure.

3.1 Offshore Wind Turbines Built under ABS Survey (1 July 2020)

Offshore wind turbines built and constructed to the satisfaction of the ABS Surveyors and to the requirements of this Guide or to their equivalent, where approved by ABS, may be classed and distinguished in the ABS *Record* by the following symbol:

⌘ A1 Offshore Wind Turbine (Bottom-Founded)

The mark ⌘ (Maltese cross) signifies that the offshore wind turbine was built, installed and commissioned to the satisfaction of the ABS Surveyors.

3.3 Offshore Wind Turbines Not Built under ABS Survey (1 July 2020)

Offshore wind turbines not built under ABS survey but submitted for classification, are subject to special classification survey. Where found satisfactory and thereafter approved by ABS, such installations may be classed with the symbol:

A1 Offshore Wind Turbine (Bottom-Founded)

5 Additional Class Notations (1 July 2020)

5.1 Notation for Rotor-Nacelle Assembly (RNA) and Tower (1 July 2020)

The RNA and the tower, as defined in 1-2/3.43 and 1-2/3.51.1, may be considered for the classification at the Owner's request. A Bottom-founded Offshore Wind Turbine may be classed and distinguished in the *Record* by the optional class notation **RNA** if

- i) The RNA and the tower are in compliance with the ABS Type Approval requirements in the ABS *Rules for Conditions of Classification – Offshore Units and Structures (Part I)*, and
- ii) The site-specific assessment demonstrates, as a minimum, that loads and deflections calculated for the Bottom-founded Offshore Wind Turbine under the site-specific conditions do not exceed those calculated for the RNA and the tower approved by the ABS Type Approval.

5.3 Design Fatigue Life (1 July 2020)

5.3.1 New Construction

The design fatigue life value notation is to be assigned and published in the *Record*. The “design fatigue life” refers to the target value set by the owner or designer, not the value calculated in the analysis. The design fatigue life is to be verified to be in compliance with the fatigue criteria in this Guide. The class notation **FL(number of years), Year** will be assigned to identify the design fatigue life in years and the year of maturation of fatigue life in the defined site. For example, **FL(25), 2045** for a wind turbine installation built in 2020 if the minimum design fatigue life specified is 25 years.

The class notation **FL(number of years), Year** is mandatory to be assigned and published in the ABS *Record*.

5.3.2 Life Extension of Bottom-founded Offshore Wind Turbines

When the operating life of a Bottom-founded Offshore Wind Turbine exceeds the design fatigue life specified in the **FL (number of years), Year** notation for which it was classed, an evaluation is to be made and appropriate actions are to be taken to extend the use of the Bottom-founded Offshore Wind Turbine up to the new operating life under the site-specific conditions.

If the environmental criteria for the site or operating area have been revised since the original approval due to new environmental data or changing environmental conditions, the coastal State may require the use of new environmental criteria for the life extension.

For the life extension of a Bottom-founded Offshore Wind Turbine, the following actions will be required:

- Structural strength analysis
- Design review and surveys related to any major modifications affecting class items.
- Verification from the original fatigue analysis that the actual fatigue values of all the structural elements are still higher than the proposed extended fatigue life; or
- New fatigue analysis covering all the structural elements in accordance with **FL** requirements, as applicable.
- Identification of structural elements or details with a fatigue life below the new intended design fatigue life and proposed actions to increase the fatigue life of those elements or details.
- Design review and surveys of structural modifications proposed as a result of the fatigue analysis.
- Enhanced survey program to monitor those structural elements or details with lower fatigue life which cannot be modified or renewed on site.
- Special survey, including underwater inspection, to determine the structural condition at the time of the life extension.

Survey requirements for extension of use are given in 7-2/1.21.

After the life extension is completed in accordance with this Guide, the existing **FL** notation with total number of fatigue life and year of maturation is to be updated accordingly.

7 Rules for Classification

7.1 Application (1 July 2020)

The requirements in this Guide are applicable to the Bottom-founded Offshore Wind Turbine, as defined in 1-2/3.3. The design criteria specified in this Guide are intended for the Bottom-founded Offshore Wind Turbine to achieve the normal safety class as defined in IEC 61400-3-1 (2019).

This Guide is applicable to those features that are permanent in nature and can be verified by plan review, calculation, physical survey or other appropriate means. Any statement in this document regarding other features is to be considered as guidance to the designer, Fabricator, and Owner.

7.3 References (1 July 2020)

References are made in this Guide to ABS Rules and other criteria issued by ABS and other organizations. Appendix 4 contains a list of such references. Unless otherwise noted, the applicable edition of a reference is the one officially issued and available on the date the Agreement for Classification is accepted by ABS. Where an edition or date associated with a reference is given, it means that particular edition is relevant to the topic being presented in this Guide. ABS may consider at its discretion, upon the request of the Owner, the application of other appropriate alternative methods and recognized codes of practice.

7.5 Alternatives (1 July 2020)

Any departure from the requirements of this Guide may be considered by ABS on the basis of suitable engineering analyses or risk assessments. In the case of such departures, classification is subject to ABS's approval upon a demonstration of acceptable levels of safety in line with the principles of this Guide and recognized and generally accepted current offshore wind industry practice. Risk acceptance criteria are subject to approval by ABS. See the *ABS Guidance Notes on Risk Assessment Application for the Marine and Offshore Industries* contains an overview of risk assessment techniques and additional information.

Using a risk assessment approach to justify alternatives may be applicable either to the Bottom-founded Offshore Wind Turbine as a whole or to individual systems, subsystems or components. As appropriate, remote hazards outside of the bounds of the system under consideration **must be taken into account**. Such account must include incidents relating to remote hazards directly affecting or being influenced by the system under consideration. ABS will consider the application of risk-based techniques in the design of the Bottom-founded Offshore Wind Turbine as well as surveys during construction and surveys for maintenance of class.

Portions of the Bottom-founded Offshore Wind Turbine not included in the risk assessment are to comply with the applicable parts of the ABS Rules.

The following are the responsibility of the Owner:

- i) Risk acceptance criteria
- ii) Hazard identification
- iii) Risk assessment
- iv) Risk management
- v) Compliance with the applicable requirements of the coastal State or other governmental authorities

9 Design Documentation to be Submitted (1 July 2020)

The design documentation to be submitted is to describe the data, tools, procedures and methodologies of design and analysis which are employed to establish the design of the Bottom-founded Offshore Wind Turbine. The intended design life is also to be stated.

Where model testing is used as a basis for a design, the applicability of the test results depends on the demonstration of the adequacy of the methods employed, including enumeration of possible sources of error, limits of applicability, and methods of extrapolation to full scale. Preferably, procedures are to be reviewed and agreed upon before model testing is done.

As required in the subsequent Paragraphs, calculations are to be submitted to demonstrate the sufficiency of the proposed design. Such calculations are to be presented in a logical and well-referenced fashion employing a consistent system of units. Where the calculations are in the form of computer analysis, the submittal is to provide input and output data with computer generated plots for the structural model. A program description (not code listings), user manuals, and the results of program verification sample problems may be required to be submitted.

The design documentation to be submitted is to include the reports, calculations, plans, specifications and other documentation where applicable. The extensiveness of the submitted documentation is to reflect:

- i) The uniqueness of a specific design of the Bottom-founded Offshore Wind Turbine within an offshore wind farm, and
- ii) The level of experience with conditions in an area where the Bottom-founded Offshore Wind Turbine is to be located.

Design documentation should generally be submitted electronically to ABS. However, hard copies will also be accepted.

9.1 Reports

Reports by consultants and other specialists used as a basis for design are to be submitted for review. The contents of reports on offshore wind farm conditions, environmental considerations, foundation data, and materials are, in general, to comply with the recommended list of items given below.

9.1.1 Offshore Wind Farm Conditions (1 July 2020)

A report on offshore wind farm conditions is to present the configuration of an offshore wind farm and exact locations of all individual offshore wind turbines, transformer platform, service and accommodation units and any other supporting structures and facilities in the offshore wind farm where applicable.

The report is also to contain information on wind turbine RNAs, particularly those properties that are used as the input or as the basis of the input for the load calculation and structural assessment for the Support Structure.

9.1.2 Environmental Considerations (1 July 2020)

Reports on environmental considerations, as described in Chapter 3, Section 1, are to describe all environmental phenomena appropriate to the areas of construction, transportation, installation, maintenance, and repair. The types of environmental phenomena to be accounted for, as appropriate to the type and location of the Bottom-founded Offshore Wind Turbine, include wind, waves, current, temperature, tide, marine growth, chemical components and density of air and water, snow and ice, earthquake, and other pertinent phenomena.

The establishment of environmental conditions is to be based on appropriate original data or, when permitted, data from analogous areas. Demonstrably valid statistical models are to be employed to perform the extrapolation to long-term values. Calculations required to establish the pertinent design environmental parameters are to be submitted for review.

The report on environmental considerations is also to contain the calculations which quantify the effects or loadings on the Bottom-founded Offshore Wind Turbine if they are not provided in other documentation.

9.1.3 Foundation Data (1 July 2020)

Reports on foundation data are to present the **findings** of investigations or, when applicable, data from analogous areas on geophysical, geological and geotechnical considerations existing at and near the installation site of an offshore wind turbine. The manner in which such data is established and the specific items to be assessed are to comply with 5-4/1 and 5-4/5. The report is to contain a listing of references to cover the investigation, sampling, testing, and interpretive techniques employed during and after the site investigation.

The report is to include a **description** of the predicted soil-structure interaction, such as p-y data, to be used in the design. As appropriate to the planned structure, the items which may be covered in the reports are: axial and lateral pile capacities and response characteristics, the effects of cyclic loading on soil strength, scour, settlements and lateral displacements, dynamic interaction between soil and structure, the capacity of pile groups, slope stability, bearing and lateral stability, soil reactions on the structure, and penetration resistance.

Recommendations relative to any special anticipated problem regarding installation are to be included in the report. Items such as the following are to be included, as appropriate: hammer sizes, soil erosion during installation, bottom preparation, and procedures to be followed in the case that pile installation procedures significantly deviate from those anticipated.

The documentation for the foundation design is to be submitted in accordance with Section 5-4.

9.1.4 Materials and Welding (1 July 2020)

Reports on structural materials and welding may be required for metallic structures, concrete structures or welding procedures where materials or procedures do not conform to those provided in Chapter 2 of this Guide.

For metallic structures, **where** it is intended to employ new alloys not defined by a recognized specification, reports are to be submitted indicating the adequacy of the material's metallurgical

properties, fracture toughness, yield and tensile strengths, and corrosion resistance, with respect to their intended application and service temperatures.

For concrete gravity foundation structures, when it is not intended to test or define material properties in accordance with applicable standards of the American Society for Testing and Materials (ASTM) as listed in Chapter 2, Section 1 of this Guide, a report is to be provided indicating the standards actually to be employed and their relative adequacy with respect to the corresponding ASTM standards.

9.3 Design Data and Calculations (1 July 2020)

Design and analysis calculations are to be submitted for items relating to loadings and structural responses for in-place and marine operations, structural dynamic properties, and foundation designs. Calculations are in general to include the items listed below.

Calculations **to be provided** in association with environmental considerations and foundation data are described in 1-1/9.1.

9.3.1 Loadings

Calculations for loadings are to be submitted in accordance with Chapter 4.

9.3.2 Structural Dynamic Properties (15 January 2013)

Calculations of natural periods of the Support Structure are to be submitted for review. A resonance diagram (Campbell diagram) depicting the relationship between the rotor speeds and the natural periods of turbine components and the Support Structure is to be submitted.

As applicable, the calculation of vibration amplitudes, velocities, and accelerations of the Support Structure may also be required.

9.3.3 Structural Responses (1 July 2020)

The stress and deflection calculations to be submitted are to include nominal element or member stresses and deflections. Calculations are also required for stresses in localized areas and structural joints, dynamic responses of the structure, and fatigue life of critical members and joints.

For a pile-supported structure, calculations for stresses in each pile and the load capacity of the connection between the structure and the pile are to be submitted. For a gravity structure, calculations are to be submitted for the effects of soil's reaction on the foundation structure.

For a Self-Elevating Unit to be used as the **Substructure** of an offshore wind turbine, the applicable calculations required in 3-1-2/3 of the *ABS Rules for Building and Classing Mobile Offshore Units* are to be submitted.

9.3.4 Marine Operations (1 July 2020)

Calculations are to be submitted in compliance with **Section 5-5 of this Guide**. For structural responses resulting from the marine operations described in **Section 5-5**, calculations are to demonstrate the adequacy of structural elements, members, or local structure, as deemed necessary. In addition, the calculations are to demonstrate, as applicable, that deflections resulting from applied loadings and overall structural displacement and settlement do not impair the structural performance of the Support Structure.

9.3.5 Other Calculations (1 July 2020)

For the Bottom-founded Offshore Wind Turbine carrying the optional **RNA** notation, a site-specific assessment is to be performed to calculate the loads and deflections of the type approved RNA and tower under site-specific conditions.

As required, additional calculations which demonstrate the adequacy of an overall design **may be required**. Such calculations are to include those performed in the design of corrosion protection system.

9.5 Plans and Specifications (1 July 2020)

Plans or specifications depicting or describing arrangements and details of major items of the Bottom-founded Offshore Wind Turbine are to be submitted for review or approval in a timely manner.

Where deemed appropriate, and when requested by the Owner, a schedule for information submittal and plan approval can be jointly established by the Owner and ABS. This schedule, which affects the time required for review of submitted data and ABS will adhere to as far as reasonably possible, is to reflect the construction schedule and the complexity of the Bottom-founded Offshore Wind Turbine.

These plans are to include the following, where applicable.

- i) Arrangement plans, elevations, and plan views clearly showing in sufficient detail the overall configuration, dimensions and layout of the rotor, nacelle, tower, substructure, foundation, etc.
- ii) Layout plans indicating the locations, dimension and weights of main turbine components (e.g., blade, hub, shaft, nacelle, etc.) and the components (e.g., electrical, mechanical and control systems, etc.) in nacelle housing.
- iii) Layout of secondary structures, fenders, ladders, access platform, boat landing, J-tube, etc.
- iv) Structural plans indicating the complete structural arrangement, dimensions, member sizes, plating and framing, material properties, and details of connections and attachments
- v) **Tower scantlings and tower-RNA connection details, where applicable**
- vi) **Tower-hull connection details**
- vii) **Tower pedestal and foundation (transit piece supporting the tower) details**
- viii) Pile plans indicating arrangements, nominal sizes, thicknesses and penetration
- ix) Welding details and procedures, and schedule of nondestructive testing
- x) Corrosion control systems
- xi) Various information in support of novel features utilized in the design where applicable
- xii) **In-Service Inspection Plan**

For a Self-Elevating Unit to be used as the **Substructure** of an offshore wind turbine, the additional plans and design data required in 3-1-2/1 of the *ABS Rules for Building and Classing Mobile Offshore Units* are to be submitted.

9.7 Information Memorandum (1 July 2020)

An information memorandum is to be prepared and submitted to ABS. ABS will review the contents of the memorandum to establish consistency with other data submitted for the purpose of obtaining classification. ABS will not review the contents of the memorandum for their accuracy or the features described in the memorandum for their adequacy.

An information memorandum is to contain, as appropriate to the installation, the following:

- i) Specifications of the RNA **and the tower suitable** to be installed **for the site**
- ii) **Valid wind turbine type certificate of the RNA and the tower**
- iii) **Description of the wind turbine control system concept and the control system (structure of the control system, sequences of the start and stop procedures, behavior of the turbine during normal operation and on detection of malfunctions, statement of trigger criteria)**

- iv) Description of turbine control system fault modes and frequency and duration of fault events
- v) Description of the procedure for manual intervention after the activation of any wind turbine protection functions
- vi) Description of the wind turbine braking systems and their behavior (structure of the braking systems, mode of operation, characteristic quantities, time constants)
- vii) Functional description of the wind turbine locking devices
- viii) Description of the wind turbine control software used for load simulation (interfaces to the load analysis, such as program modules for blade pitching, as well as identification of critical load cases)
- ix) Description of the process that confirms that the control system (including software) on the wind turbine has the same behavior as modeled in the load simulation
- x) Computer software interface of the turbine control system linked to the load analysis program if an independent analysis is requested
- xi) Site plan indicating general features at the installation site and the layout of the offshore wind farm
- xii) Environmental design criteria, including the recurrence interval used to assess environmental phenomena
- xiii) Plans showing the general arrangement of the Support Structure and its connection to the RNA
- xiv) Description of the safety and protective systems provided
- xv) Description of the modes of operation
- xvi) Description of the export or inter-array power cable termination at the substructure
- xvii) Listing of governmental authorities having cognizance over the offshore wind farm
- xviii) Listing of any novel features
- xix) Brief description of any monitoring proposed and security systems installed on the Support Structure
- xx) Description of transportation, installation and maintenance procedures

11 Operating Manual (1 July 2020)

The Operating Manual of the Bottom-founded Offshore Wind Turbine is to be submitted for review by ABS solely to verify the operational procedures and conditions are consistent with the design information, criteria and limitations considered in the classification. ABS is not responsible for the operation of offshore wind turbines.

The Operating Manual is to be retained by the **Owner or operator** and made readily available to the ABS Surveyor and to service personnel prior to conducting any maintenance or inspection.

The Operating Manual required by this Subsection does not need to be in addition to that required by the coastal State or other governmental authorities.

CHAPTER 1 Conditions of Classification

SECTION 2 Definitions

1 Types of Substructure (1 July 2020)

1.1 Pile-Supported Substructure (1 July 2020)

A structure supported by foundation elements or suction type piles driven into the sea floor.

1.3 Gravity Structure (1 July 2020)

A structure supported by the foundation structure resting directly on the sea floor

The geometry and weight of foundation structure are to be selected to **utilize** available cohesive and frictional strength components of the sea floor soil to resist loadings.

1.5 Self-Elevating Unit to be Used as the Substructure (1 July 2020)

A unit with movable legs capable of raising its hull above the surface of the sea and intended to be used as the site-dependent **substructure** of an offshore wind turbine.

The hull of a self-elevating unit has sufficient buoyancy to transport the unit to a desired location. Once on location, the hull is raised to a predetermined elevation above the sea surface on its legs, which are supported by the sea floor. The legs of such unit may be designed to penetrate the seabed, may be fitted with enlarged sections or footings, or may be attached to a bottom mat.

1.7 Compliant Tower

A structure that consists of a slender tower supported at the sea floor by an installed foundation or by a large spud can.

A compliant tower may also be partially supported by buoyancy aids. Guy lines may be used for lateral restraints.

3 Terminology

3.1 Material Application Categories (1 July 2020)

The material application of structural members in the Bottom-founded Offshore Wind Turbine is to be in accordance with the categories listed in this paragraph.

3.1.1 Special Application Structure

Special application structure refers to highly stressed members, located at intersections of main structural elements and other areas of high stress concentration where the occurrence of a fracture could induce a major structural failure.

3.1.2 Primary Application Structure

Primary application structure refers to primary load carrying members of a structure where the occurrence of a fracture could induce a major structural failure.

3.1.3 Secondary Application Structure

Secondary application structure refers to less critical members due to a combination of lower stress and favorable geometry or where an incidence of fracture is not likely to induce a major structural failure.

3.3 Bottom-founded Offshore Wind Turbine (1 July 2020)

A Bottom-founded Offshore Wind Turbine contains a site-dependent Substructure (see 1-2/3.51.2) and its Foundation (see 1-2/3.51.3) supporting the wind turbine RNA and the tower.

The design of the Bottom-founded Offshore Wind Turbine is based on environmental, electrical network, and foundation conditions at a particular offshore site where it is intended to be installed. The sea floor attachment of the Substructure may be obtained by pilings, direct bearing, or other types of foundation.

3.5 Consultant (1 July 2020)

Any person who, through education and experience, has established credentials of professionalism and knowledge in the stated field.

3.7 Cut-In Wind Speed (V_{in})

The lowest 10-minute mean wind speed at Hub Height at which the wind turbine starts to produce power in the case of steady wind without turbulence

3.9 Cut-Out Wind Speed (V_{out}) (15 January 2013)

The highest 10-minute mean wind speed at Hub Height at which the wind turbine is designed or chosen to produce power in the case of steady wind without turbulence.

3.11 Emergency Stop (1 July 2020)

Rapid shutdown of the wind turbine triggered by manual intervention

3.13 Fabricator (15 January 2013)

A Fabricator is any person or organization having the responsibility to perform any or all of the following: fabrication, assembly, erection, inspection, testing, load-out, transportation, and installation.

3.15 Gust

Brief rise and fall in wind speed lasting less than 1 minute

3.17 Hub Height (15 January 2013)

Height of the center of the swept area of the wind turbine rotor above the Still Water Level.

3.19 Idling

Condition of a wind turbine that is rotating slowly and not producing power

3.21 Mean Sea Level or Mean Still Water Level (MSL)

Average level of the sea over a period long enough to remove variations due to waves, tides and storm surges.

3.23 Mean Wind Speed

Statistical mean value of the instantaneous wind speed over a specified time interval

3.25 Normal Shutdown

Wind turbine shutdown operation in which all stages are under the control of the control system

3.27 Offshore Wind Farm

A group of Offshore Wind Turbines installed at an offshore site.

3.29 Offshore Wind Turbine (1 July 2020)

An Offshore Wind Turbine consists of a Rotor-Nacelle Assembly and its Support Structure, as defined in 1-2/3.43 and 1-2/3.51, respectively.

3.31 Omni-directional (Wind, Waves or Currents)

Acting in all directions

3.33 Owner

An owner is any person or organization who owns Offshore Wind Turbines.

3.35 Parked

Condition of a wind turbine that is either in a Standstill or an Idling condition, depending on the design of the wind turbine

3.37 Rated Power

Quantity of power assigned, generally by a manufacturer, for a specified operating condition of a component, device, or equipment. For wind turbines, it is the maximum continuous electrical power output which a wind turbine is designed to achieve under normal operating and external conditions.

3.39 Rated Wind Speed (V_r)

Minimum 10-minute mean wind speed at Hub Height at which a wind turbine's Rated Power is achieved in the case of steady wind without turbulence

3.41 Return Period (Recurrence Period)

A return period is an average time duration between occurrences of an event or of a particular value being exceeded. A return period in years is equal to the reciprocal of the annual probability of exceedance of an event or of a particular value of a random parameter such as wind speed, wave height or sea elevation.

3.43 Rotor-Nacelle Assembly (RNA)

A Rotor-Nacelle Assembly of a horizontal axis wind turbine, carried by the Support Structure, consists of:

- i) The Rotor components, including blades, hub, shaft, and spinner.
- ii) The Nacelle, a housing which contains the mainframe, generator frame, drive train components, electrical generator components, wind turbine control and protection components and other elements on top of the Tower.

3.45 **Splash Zone (15 January 2013)**

Part of the Support Structure containing the areas above and below the Mean Sea Level and regularly subjected to wetting due to wave and tide action.

3.47 **Standstill**

Condition of a wind turbine that is not rotating

3.49 **Still Water Level (SWL) (15 January 2013)**

Abstract water level used for the calculation of wave kinematics and wave crest elevation. Still Water Level, which can be either above or below the Mean Sea Level, is calculated by adding the effect of tide and surge to the Mean Sea Level.

3.51 **Support Structure (15 January 2013)**

A Support Structure consists of the Tower, Substructure, and Foundation, which are defined as follows:

3.51.1 **Tower**

Structure component which extends upward from somewhere above the Still Water Level and connects the Substructure to the Rotor-Nacelle Assembly (RNA)

3.51.2 **Substructure**

Structure component which extends upward from the sea floor and connects the Foundation to the Tower

3.51.3 **Foundation**

Structural and/or geotechnical component which is located on and beneath the sea floor and transfers the loads acting on the structure into the sea floor

3.53 **Surveyor (1 July 2020)**

A person employed by ABS whose principal functions are the surveillance during construction and the survey of marine structures and their components for compliance with the ABS Rules or other standards deemed suitable by ABS.

3.55 **Turbulence Intensity**

Ratio of the wind speed standard deviation to the mean wind speed, determined from the same set of measured data samples of wind speed, and taken over a specified period of time

3.57 **Uni-directional (Wind, Waves or Currents)**

Acting in a single directions

3.59 **Water Depth**

Vertical distance between the sea floor and the Still Water Level

3.61 **Wind Profile (Wind Shear Law)**

Mathematical expression for assumed wind speed variation with height above the Still Water Level

3.63 **Yawing**

Rotation of the rotor axis about a vertical axis for horizontal axis wind turbines

3.65 **Yaw Misalignment**

Horizontal deviation of the wind turbine rotor axis from the wind direction

CHAPTER 2

Materials and Welding

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CHAPTER 2 Materials and Welding

SECTION 1 Materials

1 Structural Steels

1.1 General

1.1.1 Scope

This Subsection covers specifications for materials used for the construction of offshore steel structures. When material other than steel is used as a structural material, documentation is to indicate the mechanical properties, toughness, fatigue, and corrosion characteristics of the proposed material. Where such materials are used in combination with steel, galvanic effects are to be taken into account, as applicable.

This Subsection is not intended for metals used in reinforced or prestressed concrete, which is addressed in 2-1/3. All materials are to be suitable for intended service conditions; they are to be of good quality, defined by a recognized specification and free of injurious imperfections.

1.1.2 Material Selection

Materials used are required to exhibit satisfactory formability and weldability characteristics. As required, documentation is to be submitted to substantiate the applicability of proposed steel. Reference can be made to 2-1/Table 2 and 2-1/Table 3 of this Guide for ASTM and API steel grades and to Section 3-1-3 of the *ABS Rules for Building and Classing Offshore Installations* for guidance on selection of ABS grades of steel.

1.1.3 Corrosion Control

Details of corrosion control systems (such as coatings, sacrificial anodes or impressed current systems) are to be submitted with adequate supporting data to show their suitability. Such information is to indicate the extent to which the possible existence of stress corrosion, corrosion fatigue, and galvanic corrosion due to dissimilar metals are to be considered. Where the intended sea environment contains unusual contaminants, any special corrosive effects of such contaminants are also to be considered. Appropriate coatings may be used to achieve satisfactory corrosion protection for miscellaneous parts such as bolts and nuts.

1.1.4 Toughness

Materials are to exhibit fracture toughness which is satisfactory for the intended application as supported by previous satisfactory service experience or appropriate toughness tests. Where the

presence of ice is judged as a significant environmental factor, material selection may require special consideration.

1.1.5 Through Thickness Stress

In cases where principal loads, from either service or weld residual stresses, are imposed perpendicular to the surface of a structural member, the use of steel with improved through thickness (Z-direction) properties may be required.

1.3 Steel Properties

1.3.1 General

Material specifications are to be submitted for review or approval. Due regard is to be given to established practices in the country in which material is produced and the purpose for which the material is intended.

1.3.2 Tensile Properties

In 2-1/1.5.2 TABLE 1, the designation Group I, II or III is used to categorize tensile properties.

1.3.3 Toughness

Appropriate supporting information or test data are to indicate that the toughness of the steels will be adequate for their intended application and minimum service temperature. Criteria indicative of adequate toughness are contained in 2-1/1.5.

1.3.4 Bolts and Nuts

Bolts and nuts are to have mechanical and corrosion characteristics comparable to the structural elements being joined and are to be manufactured and tested in accordance with recognized material standards.

1.5 Toughness Criteria for Steel Selection

1.5.1 General

When members are subjected to significant tensile stress, fracture toughness is to be considered in the selection of materials.

1.5.2 Steel Classification

Steels are to be classified as Groups I, II or III according to their tensile properties as listed in 2-1/ Table 1. It should be noted that the yield strengths given in 2-1/1.5.2 TABLE 1 are provided only as a means of categorizing steels.

TABLE 1
Steel Tensile Properties

Group	Yield Strength f_y	
	ksi	MPa
I	$f_y < 40$	$f_y < 275$
II	$40 < f_y \leq 60$	$275 < f_y \leq 415$
III	$60 \leq f_y \leq 100$	$415 \leq f_y \leq 690$

Some of the typical ASTM and API steels belonging to the groups of 2-1/1.5.2 TABLE 1 are shown in 2-1/1.5.2 TABLE 2 and 2-1/1.5.2 TABLE 3. Steels other than those mentioned therein

may be used, provided that their chemical composition, mechanical properties and weldability are similar to those listed.

TABLE 2
Structural Steel Plates and Shapes

<i>Group</i>	<i>Specification & Grade</i>	<i>Yield Strength</i>		<i>Tensile Strength</i>	
		<i>ksi</i>	<i>MPa</i>	<i>ksi</i>	<i>MPa</i>
I	ASTM A36	36	250	58-80	400-550
	ASTM A131 Grade A (ABS Grade A)	34	235	58-75	400-515
	ASTM A285 Grade C	30	205	55-75	380-515
	ASTM A131 Grades B, D (ABS Grades B, D)	34	235	58-75	400-515
	ASTM A516 Grade 65	35	240	65-85	450-585
	ASTM A573 Grade 65	35	240	65-77	450-530
	ASTM A709 Grade 36T2	36	250	58-80	400-550
	ASTM A131 Grade E (ABS Grade E) (ABS Grade E)	34	235	58-75	400-515

Group	Specification & Grade	Yield Strength		Tensile Strength	
		ksi	MPa	ksi	MPa
II	ASTM A572 Grade 42	42	290	60 min.	415 min.
	ASTM A572 Grade 50	50	345	65 min.	450 min.
	ASTM A588 (to 2 in. thick)	50	345	70 min.	485 min.
	ASTM A709 Grades 50T2, 50T3	50	345	65 min.	450 min.
	ASTM A131 Grade AH32 (ABS Grade AH32)	46	315	68-85	470-585
	ASTM A131 Grade AH36 (ABS Grade AH36)	51	350	71-90	490-620
	API Spec 2H-Grade 42	42	290	62-80	425-550
	API Spec 2H-Grade 50 (to 2 ¹ / ₂ in. thick)	50	345	70-90	485-620
	(over 2 ¹ / ₂ in. thick)	47	325	70-90	485-620
	API Spec 2W-Grade 42 (to 1 in. thick)	42-67	290-460	62 min.	425 min.
	(over 1 in. thick)	42-62	290-430	62 min.	425 min.
	Grade 50 (to 1 in. thick)	50-75	345-515	65 min.	450 min.
	(over 1 in. thick)	50-70	345-485	65 min.	450 min.
	Grade 50T (to 1 in. thick)	50-80	345-550	70 min.	485 min.
	(over 1 in. thick)	50-75	345-515	70 min.	485 min.
	API Spec 2Y-Grade 42 (to 1 in. thick)	42-67	290-460	62 min.	425 min.
	(over 1 in. thick)	42-62	290-430	62 min.	425 min.
	Grade 50 (to 1 in. thick)	50-75	345-515	65 min.	450 min.
	(over 1 in. thick)	50-70	345-485	65 min.	450 min.
	Grade 50T (to 1 in. thick)	50-80	345-550	70 min.	485 min.
	(over 1 in. thick)	50-75	345-515	70 min.	485 min.
	ASTM A131 Grades DH32, EH32 (ABS Grades DH32, EH32)	46	315	68-85	470-585
	Grades DH36, EH36 (ABS Grades DH36, EH36)	51	350	71-90	490-620
	ASTM A537 Class 1 (to 2 ¹ / ₂ in. thick)	50	345	70-90	485-620
	ASTM A633 Grade A	42	290	63-83	435-570
	Grades C, D	50	345	70-90	480-620
	ASTM A678 (80) Grade A	50	345	70-90	485-620
	ASTM A992	50-65	345-450	65 min.	450 min.

Group	Specification & Grade	Yield Strength		Tensile Strength	
		ksi	MPa	ksi	MPa
III	ASTM A537 Class 2	60	415	80-100	550-690
	ASTM A633 Grade E	60	415	80-100	550-690
	ASTM A678 (80) Grade B	60	415	80-100	550-690
	API Spec 2W-Grade 60 (to 1 in. thick)	60-90	415-620	75 min.	515 min.
	(over 1 in. thick)	60-85	415-585	75 min.	515 min.
	API Spec 2Y-Grade 60 (to 1 in. thick)	60-90	415-620	75 min.	515 min.
	(over 1 in. thick)	60-85	415-585	75 min.	515 min.
	ASTM A710-Grade A Class 3 (to 2 in. thick)	75	515	85 min.	585 min.
	(2 in. to 4 in. thick)	65	450	75 min.	515 min.
	(over 4 in. thick)	60	415	70 min.	485 min.

TABLE 3
Structural Steel Pipes

Group	Specification & Grade	Yield Strength		Tensile Strength	
		ksi	MPa	ksi	MPa
I	API 5L-Grade B	35	240	60 min.	415 min.
	ASTM A53 Grade B	35	240	60 min.	415 min.
	ASTM A135 Grade B	35	240	60 min.	415 min.
	ASTM A139 Grade B	35	240	60 min.	415 min.
	ASTM A381 Grade Y35	35	240	60 min.	415 min.
	ASTM A500 Grade A	33-39	230-270	45 min.	310 min.
	ASTM A501	36	250	58 min.	400 min.
	ASTM A106 Grade B	35	240	60 min.	415 min.
	ASTM A524 (strength varies with thickness)	30-35	205-240	55-85	380-585
II	API 5L95 Grade X42 (2% max. cold expansion)	42	290	60 min.	415 min.
	API 5L95 Grade X52 (2% max. cold expansion)	52	360	66 min.	455 min.
	ASTM A500 Grade B	42-46	290-320	58 min.	400 min.
	ASTM A618 Grade Ia, Ib & II (to $\frac{3}{4}$ in. thick)	50	345	70 min.	485 min.
	API 5L95 Grade X52 (with SR5, SR6, or SR8)	52	360	66 min.	455 min.

1.5.3 Toughness Characteristics

Satisfactory toughness characteristics can be demonstrated by any one of the following.

- i) Demonstration of past successful application, under comparable conditions, of a steel produced to a recognized standard (such as those of the ASTM, API or other recognized standard).

- ii) Demonstration that a steel manufactured by a particular producer using a specific manufacturing process has minimum toughness levels representative of those listed herein.
- iii) Charpy impact testing in accordance with 2-1/1.7.3 TABLE 4.

1.7 Minimum Service Temperature

Minimum service temperature refers to the temperature of the material and is generally to be established in accordance with 2-1/1.7.1 to 2-1/1.7.3 below. This temperature is to be based on meteorological data taken over a period of not less than 10 years for the lowest average daily temperature.

1.7.1 Material below the Splash Zone

For material below the splash zone (see 5-1/5.11), the service temperature is defined as 0°C (32°F). A higher service temperature may be used if adequate supporting data can be presented relative to the lowest average daily water temperature applicable to the depths involved.

1.7.2 Material within or above the Splash Zone

For material within or above the splash zone, the service temperature is the same as the lowest average daily atmospheric temperature. A higher service temperature may be used if the material above the waterline is warmed by adjacent sea water temperature or by auxiliary heating.

1.7.3 Special Conditions

In all cases where material temperature is reduced by localized cryogenic storage or other cooling conditions, such factors are to be taken into account in establishing minimum service temperature.

TABLE 4
Charpy Toughness Specification for Steels

Group	Section Size	Energy Absorption (Longitudinal)	
		ft-lb	Joules
I	6 mm < t < 19 mm (0.25 in. < t < 0.75 in.)	15	20
	t > 19 mm (0.75 in.)	20	27
II, III	t > 6 mm (0.25 in.)	25	34

Notes:

- 1 Test Temperatures – The following applies for service temperatures down to -30°C (-22°F). For lower service temperatures, test requirements are to be specially considered.

- a) For structural members and joints whose performance is vital to the overall integrity of the structure and which experience an unusually severe combination of stress concentration, rapid loading, cold working, and restraint, the impact test guidelines of 2-1/Table 4 are to be met at test temperatures as given below.

Group	Test Temperature	Minimum Service Temperature (As determined by 2-1/1.7)
I, II	30°C (54°F) below Minimum Service Temperature	
III	-40°C (-40°F)	-10°C (32°F)
	-50°C (-58°F)	-10°C (14°F)
	-50°C (-58°F)	-20°C (-4°F)
	-60°C (-76°F)	-30°C (-22°F)

- b) For structural members and joints which sustain significant tensile stress and whose fracture may pose a threat to the survival of the structure, the impact test guidelines of 2-1/Table 4 are to be met at test temperatures as given below.

Group	Test Temperature	Minimum Service Temperature (As determined by 2-1/1.7)
I, II	10°C (18°F) below Minimum Service Temperature	
III	−40°C (−40°F)	−10°C (32°F)
	−40°C (−40°F)	−10°C (14°F)
	−40°C (−40°F)	−20°C (−4°F)
	−50°C (−58°F)	−30°C (−22°F)

- c) For primary structural members subject to significant tensile stresses and whose usage warrants impact toughness testing, the impact test guidelines of 2-1/Table 4 are to be met at the following test temperatures.

Group	Test Temperature	Minimum Service Temperature (As determined by 2-1/1.7)
I, II	At Minimum Service Temperature	
III	Same as Note 1b	

- d) For structural members which have sufficient structural redundancy so that their fracture would not pose a threat to the survivability of the structure, the toughness criteria specified for c above may be relaxed provided the materials used in such cases are appropriate for the loading conditions, loading rates, and temperatures encountered in service.

- 2 Impact tests are not necessary for section sizes below 6 mm (0.25 in.) in thickness.
- 3 Energy values in 2-1/Table 4 are minimum average values for full-size longitudinal specimens. Alternative toughness criteria which may be applied are:
 - i) Under-sized longitudinal specimens: proportional reduction in 2-1/1.7.3 TABLE 4 energy values in accordance with ASTM A20 or equivalent.
 - ii) Transverse specimens: $\frac{2}{3}$ of the energy values shown in 2-1/1.7.3 TABLE 4 but in no case less than 20 Joules (15 ft-lb).
 - iii) Longitudinal or transverse specimens: lateral expansion is not to be less than 0.5 mm (0.02 in.), or 0.38 mm (0.015 in.), respectively.
 - iv) Nil-ductility temperature (NDT) as determined by drop weight tests is to be 5°C (10°F) below the test temperature indicated in note 1 above.
 - v) Other fracture toughness tests as appropriate.
- 4 The minimum number of specimens to be tested per heat is to be three; however, this number is to be increased in accordance with usage of the material (see ASTM A673 or equivalent).

3 Materials for Concrete Construction

3.1 General

3.1.1 Scope (1 July 2020)

This Subsection covers specifications for materials for concrete used in the construction of the Support Structure of offshore wind turbines. It includes the metals used in reinforced or prestressed concrete. All materials are to be suitable for intended service conditions and are to be

of good quality, defined by recognized specifications and free of injurious defects. Materials used in the construction of concrete structures are to be selected with due attention given to their strength and durability in the marine environment. Materials which do not conform to the requirements of this Subsection may be considered for approval upon presentation of sufficient evidence of satisfactory performance.

3.1.2 Zones

Particular attention is to be given in each of the following zones (see 5-1/5.9) :

- i) *Submerged zone*: chemical deterioration of the concrete, corrosion of the reinforcement and hardware, and abrasion of the concrete.
- ii) *Splash zone*: freeze-thaw durability, corrosion of the reinforcement and hardware, chemical deterioration of the concrete, and fire hazards.
- iii) *Ice zone*: freeze-thaw durability, corrosion of the reinforcement and hardware, chemical deterioration of the concrete, fire hazards, and abrasion of the concrete.
- iv) *Atmospheric zone*: freeze-thaw durability, corrosion of reinforcement and hardware, and fire hazards.

3.3 Cement

3.3.1 Type

Cement is to be equivalent to Types I or II Portland cement as specified by ASTM C150 or Portland-pozzolan cement as specified by ASTM C595. ASTM C150 Type III Portland cement may be specially approved for particular applications.

3.3.2 Tricalcium Aluminate

The tricalcium aluminate content of the cement is generally to be in the 5% to 10% range.

3.3.3 Oil Storage

For environments which contain detrimental sulfur bearing materials (such as where oil storage is planned and the oil is expected to contain sulfur compounds which are detrimental to concrete durability), the maximum content of tricalcium aluminate is to be at the lower end of the 5% to 10% range. Alternatively, pozzolans, or pozzolans and fly ash, may be added or a suitable coating employed to protect the concrete.

3.5 Water

3.5.1 Cleanliness

Water used in mixing concrete is to be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials or other substances that may be deleterious to concrete or steel.

3.5.2 Non-potable Water

If non-potable water is proposed for use, the selection of proportions of materials in the concrete is to be based on test concrete mixes using water from the same source. The strength of mortar test cylinders made with non-potable water is not to be less than 90% of the strength of similar cylinders made with potable water. Strength test comparisons are to include 7-day and 28-day strength data on mortars prepared and tested in accordance with recognized standards such as ASTM C109.

3.7 Chloride or Sulfide Content

Water for structural concrete or grout is to not contain more than 0.07% chlorides as Cl by weight of cement, nor more than 0.09% sulfates as SO_4 when tested by ASTM D512. Chlorides in mix water for prestressed concrete or grout are to be limited to 0.04% by weight of cement.

Total chloride content, as Cl, of the concrete prior to exposure is not to exceed 0.10% by weight of the cement for normal reinforced concrete and 0.06% by weight of cement for prestressed concrete.

3.9 Aggregates

3.9.1 General

Aggregates are to conform to the requirements of ASTM C33 or equivalent. Other aggregates may be used if there is supporting evidence that they produce concrete of satisfactory quality. When specially approved, lightweight aggregates similar to those specified in ASTM C330 may be used for conditions that do not pose durability problems.

3.9.2 Washing

Marine aggregates are to be washed with fresh water before use to remove chlorides and sulfates so that the total chloride and sulfate content of the concrete mix does not exceed the limits defined in 2-1/3.7.

3.9.3 Size

The maximum size of the aggregate is to be such that the concrete can be placed without voids. It is recommended that the maximum size of the aggregate is to not be larger than the smallest of the following: one-fifth of the narrowest dimension between sides of forms; one-third of the depth of slabs; three-fourths of the minimum clear spacing between individual reinforcing bars, bundles of bars, prestressing tendons or post-tensioning ducts.

3.11 Admixtures

3.11.1 General

The admixture is to be shown capable of maintaining essentially the same composition and performance throughout the work as the product used in establishing concrete proportions. Admixtures containing chloride ions are not to be used if their use will produce a deleterious concentration of chloride ions in the mixing water.

3.11.2 Recognized Standards

Admixtures are to be in accordance with applicable recognized standards such as ASTM C260, ASTM C494, ASTM C618 or equivalents.

3.11.3 Pozzolan Content

Pozzolan or pozzolan and fly ash content is not to exceed 15% by weight of cement unless specially approved.

3.13 Steel Reinforcement

Steel reinforcement used in offshore concrete structures is to be suitable for its intended service and in accordance with recognized standards.

3.13.1 Reinforcement for Non-Prestressed Concrete

Non-prestressed reinforcement is to be in accordance with one of the following specifications or its equivalents.

- i) Deformed reinforcing bars and plain bars: ASTM A615
- ii) Bar and rod mats: ASTM A184
- iii) Plain wire for spiral reinforcement: ASTM A82, ASTM A704
- iv) Welded plain wire fabric: ASTM A185
- v) Deformed wire: ASTM A496
- vi) Welded deformed wire fabric: ASTM A497

3.13.2 Welded Reinforcement

Reinforcement which is to be welded is to have the properties needed to produce satisfactory welded connections. Welding is to be in accordance with recognized specifications such as AWS D1.1 or is to be proven to produce connections of satisfactory quality.

3.13.3 Steel Reinforcement for Prestressed Concrete

Steel reinforcement for prestressed concrete is to be in accordance with one of the following specifications or equivalent.

- i) Seven-wire strand: ASTM A416
- ii) Wire: ASTM A421

3.13.4 Other Materials

Other prestressing tendons may be approved upon presentation of evidence of satisfactory properties.

3.15 Concrete

The concrete is to be designed to assure sufficient strength and durability. A satisfactory method for quality control of concrete is to be used which is equivalent to ACI 318. Mixing, placing and curing of concrete are to conform to recognized standards.

3.17 Water-Cement Ratios

Unless otherwise approved, water-cement ratios and 28-day compressive strengths of concrete for the three exposure zones are to be in accordance with 2-1/3.17 TABLE 5.

TABLE 5
Water-Cement Ratios and Compressive Strengths

<i>Zone</i>	<i>Maximum w/c Ratio</i>	<i>Minimum 28-day Cylinder Compressive Strength</i>
Submerged	0.45	35 MPa (5000 psi)
Splash and Atmospheric	0.40 to 0.45*	35 MPa (5000 psi)

* Depending upon severity of exposure

3.19 Other Durability Requirements

3.19.1 Cement Content

Minimum cement content is to verify an adequate amount of paste for reinforcement protection and generally be not less than 355 kg/m³ (600 lb/yd³).

3.19.2 Freeze-Thaw Durability

When freeze-thaw durability is required, the concrete is to contain entrained air in accordance with a recognized standard such as ACI 211.1. Attention is to be paid to the appropriate pore distribution of the entrained air and the spacing between pores in the hardened concrete. The calculated spacing factors are not to exceed 0.25 mm (0.01 in.).

3.19.3 Scouring (1 July 2020)

When severe scouring action is expected, the coarse aggregate is to be as hard as the material causing the abrasion. Also, the sand content of the concrete mix is to be kept as low as possible, and air entrainment is to be limited to the minimum appropriate to the application.

3.21 Grout for Bonded Tendons

3.21.1 General

Grout for bonded tendons is to conform to ACI 318 or equivalent.

3.21.2 Chlorides and Sulfates

Grout is not to contain chlorides or sulfates in amounts which are detrimental to the structure. Limitations are included in 2-1/3.7.

3.21.3 Contents

Grout is to consist of Portland cement and potable water, or Portland cement, sand, and potable water. Admixtures may be used only after sufficient testing to indicate that their use is beneficial and that they are free of harmful quantities of chlorides, nitrates, sulfides, sulfates or any other material which has been shown to be detrimental to the steel or grout.

3.21.4 Sand

Sand, if used, is to conform to ASTM C144 or equivalent, except that gradation may be modified as necessary to obtain increased workability.

3.21.5 Preparation

Proportions of grouting materials are to be based on results of tests on fresh and hardened grout prior to beginning work. The water content is to be the minimum necessary for proper placement but in no case more than 50% of the content of cement by weight. Grout is to be properly mixed and screened.

3.21.6 Temperature

Temperature of members at the time of grouting is to be above 10°C (50°F) and is to be maintained at this temperature for at least 48 hours.

CHAPTER 2 Materials and Welding

SECTION 2 Welding and Fabrication

1 Introduction

Welding for steel structures is to comply with the pertinent requirements of a recognized code, such as the AWS D1.1 *Structural Welding Code – Steel*, or Chapter 4 of the ABS *Rules for Materials and Welding (Part 2)*. While the requirements of this Section are to be addressed using a recognized reference code, the reference code may not provide coverage of all necessary items. Therefore, this Section provides additional requirements which, as the need arises, extend the scope of the code to make it suitable for classification purposes. Also, because of the possible wide variation of requirements which may exist among selected welding codes, 2-2/9 through 2-2/13 give some specific requirements which are intended to verify a basic degree of uniformity in the welding performed for structures classed with ABS.

3 Overview

3.1 Plans and Specifications

Submitted plans or specifications are to be in accordance with 1-1/9 and they are to indicate clearly the extent of welding for the main parts of the structure. The plans or specifications are to indicate the extent of nondestructive inspection of the weld. The welding process, filler metal and joint design are to be indicated on plans or in separate specifications submitted for approval, which are to distinguish between manual and automatic welding. The Surveyor is to be informed of the planned sequences and procedures to be followed in the erection and welding of the main structural members. In all instances, welding procedures and filler metals are to be applied which will produce sound welds that have strength and toughness comparable to that of the base material.

3.3 Workmanship and Supervision

It is to be demonstrated that all welders and welding operators to be employed in the construction of structures to be classed are properly qualified and are experienced in the type of work proposed and in the proper use of the welding processes and procedures to be followed. A sufficient number of skilled supervisors is to be employed to verify thorough supervision and control of all welding operations. Inspection of welds employing methods outlined in 2-2/7.17 is to be carried out to the satisfaction of the Surveyor.

3.5 Welding Procedures (1 July 2020)

Procedures for the welding of all joints, including types of electrodes, edge preparations, welding techniques and proposed positions, are to be established before construction begins. Details of proposed

welding procedures and sequences may be required to be submitted for review, depending on the intended application.

Special precautions **are to be taken for welding thick sections** with regard to joint preparation, preheat, welding sequence, heat input and interpass temperature. Ultrasonic inspection **may be called for** to verify the absence of injurious laminations for material used where through thickness (Z-direction) properties are important.

5 Preparation for Welding

5.1 Edge Preparation and Fitting (1 July 2020)

Edge preparations are to be accurate and uniform and the parts to be welded are to be fitted in accordance with the approved joint detail. All means **employed** for correcting improper fitting are to be to the satisfaction of the Surveyor. Where excessive root openings are encountered for butt weld connections, weld buildup of the edges may be approved by the Surveyor, depending upon the location of the joint and the welding procedures employed. Unless specially approved, such buildup of each edge, where permitted, is not to exceed $t/2$ or 12.5 mm ($1/2$ in.), whichever is less, where t is the thickness of the thinner member being welded. Where sections to be butt welded differ in thickness and have an offset on any side of more than 3 mm ($1/8$ in.), a suitable transition taper is to be provided. In general, the transition taper length is to be not less than three times the offset. The transition may be formed by tapering the thicker member or by specifying a weld joint design which will provide the required transition.

5.3 Alignment

Means are to be provided for maintaining the members to be welded in correct position and alignment during the welding operation. In general, strongbacks or other appliances used for this purpose are to be so arranged as to allow for expansion and contraction during production welding. The removal of such items is to be carried out to the satisfaction of the Surveyor.

5.5 Cleanliness

All surfaces to be welded are to be free from moisture, grease, loose mill scale, excessive rust and paint. Primer coatings of ordinary thicknesses, thin coatings of linseed oil or equivalent coatings may be used, provided it is demonstrated that their use has no adverse effect on the production of satisfactory welds. Slag and scale are to be removed not only from the edges to be welded but also from each pass or layer before the deposition of subsequent passes or layers. Weld joints prepared by arc-air gouging may require additional preparation by grinding or chipping and wire brushing prior to welding, to minimize the possibility of excessive carbon on the surfaces. Compliance with these cleanliness requirements is of prime importance in the welding of higher-strength steels (see 2-1/1.5, especially those which are quenched and tempered).

5.7 Tack Welds

Tack welds of consistent good quality, made with the same grade of filler metal as intended for production welding and deposited in such a manner as not to interfere with the completion of the final weld, need not be removed, provided they are found upon examination to be thoroughly clean and free from cracks or other defects. Preheat may be necessary prior to tack welding when the materials to be joined are highly restrained. Special consideration is to be given to using the same preheat as specified in the welding procedure when tack welding higher-strength steels, particularly those materials which are quenched and tempered. These same precautions are to be followed when making any permanent welded markings.

5.9 Run-on and Run-off Tabs

When used, run-on and run-off tabs are to be designed to minimize the possibility of high stress concentrations and base-metal and weld-metal cracking.

7 Production Welding

7.1 Environment

Proper precautions are to be taken to verify that all welding is done under conditions where the welding site is protected against the deleterious effects of moisture, wind, and severe cold.

7.3 Sequence

Welding is to be planned to progress symmetrically so that shrinkage on both sides of the structure will be equalized. The ends of frames and stiffeners are to be left unattached to the plating at the subassembly stage until connecting welds are made in the intersecting systems of plating, framing and stiffeners at the erection stage. Welds are not to be carried across an unwelded joint or beyond an unwelded joint which terminates at the joint being welded, unless specially approved.

7.5 Preheat and Postweld Heat Treatment

The use of preheat is to be considered when welding higher-strength steels, materials of thick cross section, materials subject to high restraint, and when welding is performed under high humidity conditions or when the temperature of the steel is below 0°C (32°F). The control of interpass temperature is to be specially considered when welding quenched and tempered higher-strength steels. When preheat is used, the temperature is to be in accordance with the accepted welding procedure. Postweld heat treatment, when specified, is to be carried out using an approved method.

7.7 Low-hydrogen Electrodes or Welding Processes

Unless otherwise approved, the use of low-hydrogen electrodes or welding processes is required for welding all higher-strength steels, and may also be considered for ordinary-strength steel weldments subjected to high restraint. When using low-hydrogen electrodes or processes, proper precautions are to be taken to verify that the electrodes, fluxes and gases used for welding are clean and dry.

7.9 Back Gouging

Chipping, grinding, arc-air gouging or other suitable methods are to be employed at the root or underside of the weld to obtain sound metal before applying subsequent beads for all full-penetration welds. When arc-air gouging is employed, the selected technique is to minimize carbon buildup and burning of the weld or base metal. Quenched and tempered steels are not to be flame gouged using oxy-fuel gas.

7.11 Peening

The use of peening is not recommended for single-pass welds and the root or cover passes on multipass welds. Peening, when used to correct distortion or to reduce residual stresses, is to be effected immediately after depositing and cleaning each weld pass.

7.13 Fairing and Flame Shrinking

Fairing by heating or flame shrinking, and other methods of correcting distortion or defective workmanship in fabrication of main strength members and other members which may be subject to high stresses, are to be carried out only with the expressed approval of the Surveyor. These corrective measures are to be kept to an absolute minimum when higher-strength quenched and tempered steels are involved, due to high local stresses and the possible degradation of the mechanical properties of the base material.

7.15 Weld Soundness and Surface Appearance

Production welds are to be sound, crack-free and reasonably free from lack of fusion or penetration, slag inclusions and porosity. The surfaces of welds are to be visually inspected and are to be regular and uniform with a minimum amount of reinforcement and reasonably free from undercut and overlap and free from injurious arc strikes. Contour grinding when required by an approved plan or specification or where deemed necessary by the Surveyor is to be carried out to the satisfaction of the Surveyor.

7.17 Inspection of Welds (1 July 2020)

Inspection of welded joints in important locations is to be carried out preferably by established nondestructive test methods such as radiographic, ultrasonic, magnetic-particle or dye-penetrant inspection. Approved acceptance criteria, or the *ABS Guide for Nondestructive Inspection*, are to be used in evaluating radiographs and ultrasonic indications (see also 2-2/7.21). Radiographic or ultrasonic inspection, or both, is to be used when the overall soundness of the weld cross section is to be evaluated. Magnetic-particle or dye-penetrant inspection may be used when investigating the outer surface of welds, as a check of intermediate weld passes such as root passes, and to check back chipped, ground or gouged joints prior to depositing subsequent passes. Surface inspection of important tee or corner joints in critical locations, using an approved magnetic-particle or dye-penetrant method, is to be conducted to the satisfaction of the Surveyor. Some steels, especially higher-strength steels, may be susceptible to delayed cracking. When welding these materials, the final nondestructive testing is to be delayed for a suitable period to permit detection of such defects. Weld run-on or run-off tabs may be used where practicable and these may be sectioned for examination. The practice of taking weld plugs or samples by machining or cutting from the welded structure is not recommended and is to be used only in the absence of other suitable inspection methods. When such weld plugs or samples are removed from the welded structure, the holes or cavities thus formed are to be properly prepared and welded, using a suitable welding procedure as established for the original joint.

7.19 Extent of Inspection of Welds

7.19.1 General (1 July 2020)

The minimum extent of nondestructive testing to be conducted is indicated in 2-2/7.19.4 and 2-2/7.19.5 below. The distribution of inspected welds is to be based on the classification of application of the welds, as mentioned in 2-2/7.19.3, and the variety of weld sizes used in the structure. Nondestructive testing is generally to be carried out after all forming and postweld heat treatment, and procedures are to be adequate to detect delayed cracking. Welds which are inaccessible or difficult to inspect in service are to be subject to increased levels of nondestructive inspection. Nondestructive examination of full penetration butt welds is generally to be carried out by radiographic or ultrasonic methods. Where a method (such as radiography or ultrasonics) is selected as the primary nondestructive method of inspection, the acceptance standards of such a method govern. Where inspection by any method indicates the presence of defects that could jeopardize the integrity of the structure, removal and repair of such defects are to be carried out to the satisfaction of the attending Surveyor. Should the ultrasonic method be used as the primary inspection method, such testing is to be supplemented by a reasonable amount of radiographic inspection to **confirm** adequate quality control is being achieved. To assess the extent of surface imperfections in welds made in Group III steels used in critical structural locations, representative inspection by the magnetic-particle or dye-penetrant method is to also be accomplished.

7.19.2 Plans

A plan for nondestructive testing of the structure is to be submitted. This plan is to include but is not be restricted to visual inspection of all welds, representative magnetic-particle or dye-penetrant inspection of tee and fillet welds not subject to ultrasonic inspection, and the inspection of all field welds by appropriate means. The extent and method of inspection are to be indicated on the plan, and the extent of inspection is to be based on the function of the structure and the accessibility of the welds after the structure is in service. For automated welds for which quality assurance techniques indicate consistent satisfactory performance a lesser degree of inspection may be permitted.

7.19.3 Classification of Application

Welds are to be designated as being special, primary, or secondary depending on the function and severity of service of the structure in which the welds are located. Special welds are those occurring in structural locations of critical importance to the integrity of the structure or its safe operation. Secondary welds are those occurring in locations of least importance to the overall

integrity of the structure. Primary welds are those occurring in locations whose importance is intermediate between the special and secondary classifications.

7.19.4 Extent of Nondestructive Inspection – Steel Jacket Type Structures

In general, the number of penetration type welds (i.e., butt, T, K and Y joints) to be inspected in each classification is to be based on the percentages stated below. Alternatively, the extent of radiographic and ultrasonic inspection may be based on other methods, provided the alternative will not result in a lesser degree of inspection. Where the extent of welds to be inspected is stated as a percentage, such as 20% of primary welds, this means that complete inspection of 20% of the total number of welds considered to be primary is required.

All welds considered special are to be inspected 100% by the ultrasonic or radiographic method. Twenty percent of all welds considered primary are to be inspected by the ultrasonic or radiographic method. Welds considered to be secondary are to be inspected on a random basis using an appropriate method. In locations where ultrasonic test results are not considered reliable, the use of magnetic-particle or dye-penetrant inspection as a supplement to ultrasonic inspection is to be conducted. For T, K, or Y joints, approval may be given to substituting magnetic-particle or dye-penetrant inspection for ultrasonic inspection when this will achieve a sufficient inspection quality.

Magnetic-particle or dye-penetrant inspection of fillet welds is to be accomplished for all permanent fillet welds used in jacket construction, all load-bearing connections, and all fillet welds in Special Application areas of the deck structure. The random inspection of other deck fillet welds is to be carried out at the discretion of the Surveyor.

7.19.5 Additional Inspection – Special Conditions

Additional inspection may be required depending on the type and use of the structure, the material and welding procedures involved, and the quality control procedures employed.

7.19.6 Additional Inspection – Production Experience

If the proportion of unacceptable welds becomes abnormally high, the frequency of inspection is to be increased.

7.19.7 High Through Thickness (Z-Direction) Stresses

At important intersections, welds which impose high stresses perpendicular to the member thicknesses (Z-direction loading) are to be ultrasonically inspected to assure freedom from lamellar tearing after welding.

7.21 Acceptance Criteria (1 July 2020)

As stated in 2-2/7.17 recognized acceptance criteria such as those issued by the AWS are to be employed.

When employing ABS *Guide for Nondestructive Inspection*, Class A and Class B criteria are to be applied as follows.

- i) Class A acceptance criteria are to be used for Special Application Structure and critical locations within Primary Application Structure such as circumferential welds of cylindrical and built up columns or legs, weld intersections of external plating in the Support Structure, etc.
- ii) Class B acceptance criteria are to be used for Primary Application Structure where Class A acceptance does not apply.
- iii) Twice Class B acceptance criteria are to be used for Secondary Application Structure.

When radiographic or ultrasonic inspection is specified for other types of connections, such as partial penetration and groove type tee or corner welds, modified procedures and acceptance criteria are to be specified which adequately reflect the application.

7.23 Repair Welding

Defective welds and other injurious defects, as determined by visual inspection, nondestructive test methods, or leakage under hydrostatic tests, are to be excavated in way of the defects to sound metal and corrected by rewelding, using a suitable repair welding procedure to be consistent with the material being welded. Removal by grinding of minor surface imperfections such as scars, tack welds and arc strikes may be permitted. Special precautions, such as the use of both preheat and low-hydrogen electrodes, are to be considered when repairing welds in higher-strength steel, materials of thick cross section or materials subjected to high restraint.

9 Butt Welds

9.1 Manual Welding Using Stick Electrodes

Manual welding using stick electrodes may be employed for butt welds in members not exceeding 6.5 mm ($\frac{1}{4}$ in.) in thickness without beveling the abutting edges. Members exceeding 6.5 mm ($\frac{1}{4}$ in.) are to be prepared for welding using an appropriate edge preparation, root opening and root face (land) to provide for welding from one or both sides. For welds made from both sides, the root of the first side welded is to be removed to sound metal by an approved method before applying subsequent weld passes on the reverse side. When welding is to be deposited from one side only, using ordinary welding techniques, appropriate backing (either permanent or temporary) is to be provided. The backing is to be fitted so that spacing between the backing and the members to be joined is in accordance with established procedures. Unless specially approved otherwise, splices in permanent backing strips are to be welded with full penetration welds prior to making the primary weld.

9.3 Submerged-arc Welding

Submerged-arc welding, using wire-flux combinations for butt welds in members not exceeding 16 mm ($\frac{5}{8}$ in.) in thickness, may be employed without beveling the abutting edges. Members exceeding 16 mm ($\frac{5}{8}$ in.) are normally to be prepared for welding using an appropriate edge preparation, root opening and root face (land) to provide for welding from one or both sides. When it is determined that sound welds can be made without gouging, the provisions of 2-2/7.9 are not applicable. Where the metal is to be deposited from one side only, using ordinary welding techniques, backing (either permanent or temporary) is to be provided and the members are to be beveled and fitted in accordance with established procedures.

9.5 Gas Metal-arc and Flux Cored-arc Welding

Manual semi-automatic or machine automatic gas metal-arc welding, and flux cored-arc welding using wire-gas combinations and associated processes, may be ordinarily employed utilizing the conditions specified in 2-2/9.1 except that specific joint designs may differ between processes.

9.7 Electroslag and Electrogas Welding

The use of electroslag and electrogas welding processes will be subject to special consideration, depending upon the specific application and the mechanical properties of the resulting welds and heat-affected zones.

9.9 Special Welding Techniques

Special welding techniques employing any of the basic welding processes mentioned in 2-2/9 will be specially considered, depending upon the extent of the variation from the generally accepted technique. Such special techniques include one side welding, narrow-gap welding, tandem-arc welding, open-arc welding and consumable-nozzle electroslag welding. The use of gas tungsten-arc welding will also be subject to special consideration, depending upon the application and whether the process is used manually or automatically.

11 Fillet Welds

11.1 General

The sizes of fillet welds are to be indicated on detail plans or on a separate welding schedule and are subject to approval. The weld throat size is not to be less than 0.7 times the weld leg size. Fillet welds may be made by an approved manual or automatic technique. Where the gap between the faying surfaces of members exceeds 2 mm ($\frac{1}{16}$ in.) and is not greater than 5 mm ($\frac{3}{16}$ in.), the weld leg size is to be increased by the amount of the opening. Where the gap between members is greater than 5 mm ($\frac{3}{16}$ in.), fillet weld sizes and weld procedures are to be specially approved by the Surveyor. Completed welds are to be to the Surveyor's satisfaction. Special precautions such as the use of preheat or low-hydrogen electrodes or low hydrogen welding processes may be required where small fillets are used to attach heavy members or sections. When heavy sections are attached to relatively light members, the weld size may be required to be modified.

11.3 Tee Connections

Except where otherwise indicated under 2-2/11.1, the fillet weld requirement for tee connections is to be determined by the lesser thickness member being joined. Where only the webs of girders, beams or stiffeners are to be attached, it is recommended that the unattached face plates or flanges be cut back. Except for girders of thickness greater than 25 mm (1 in.), reduction in fillet weld sizes may be specially approved in accordance with either *i*) or *ii*) specified below. However, in no case is the reduced leg size to be less than 5 mm ($\frac{3}{16}$ in.).

- i*) Where quality control facilitates working to a gap between members being attached of 1 mm (0.04 in.) or less, a reduction in fillet weld leg size of 0.5 mm (0.02 in.) may be permitted provided that the reduced leg size is not less than 8 mm ($\frac{5}{16}$ in.).
- ii*) Where automatic double continuous fillet welding is used and quality control facilitates working to a gap between members being attached of 1 mm (0.04 in.) or less, a reduction in fillet weld leg size of 1.5 mm ($\frac{1}{16}$ in.) may be permitted provided that the penetration at the root is at least 1.5 mm ($\frac{1}{16}$ in.) into the members being attached and the reduced leg size is not less than 5 mm ($\frac{3}{16}$ in.).

11.5 Lapped Joints

Lapped joints are generally not to have overlaps of less width than twice the thinner plate thickness plus 25 mm (1 in.). Both edges of an overlapped joint are to have continuous fillet welds in accordance with 2-2/11.1 or 2-2/11.7.

11.7 Overlapped End Connections

Overlapped end connections of structural members which are considered to be effective in the overall strength of the unit are to have continuous fillet welds on both edges equal in leg size to the thickness of the thinner of the two members joined. All other overlapped end connections are to have continuous fillet welds on each edge of leg sizes such that the sum of the two is not less than 1.5 times the thickness of the thinner member.

11.9 Overlapped Seams

Unless specially approved, overlapped seams are to have continuous welds on both edges of the sizes required by the approved plans and are to be in accordance with the applicable provisions of 2-2/11.1.

11.11 Plug Welds or Slot Welds (1 July 2020)

Upon consideration by ABS plug welds or slot welds may be specially approved for specific applications. Where used in the body of doublers and similar locations, such welds may be generally spaced about 300 mm (12 in.) between centers in both directions. Slot welds generally are not to be filled with weld metal. For plate thicknesses up to 13 mm ($\frac{1}{4}$ in.), fillet sizes are to be equal to plate thickness but not greater than

9.5 mm ($\frac{3}{8}$ in.); for thicknesses over 13 mm ($\frac{1}{4}$ in.) to 25 mm (1 in.) fillet sizes are to be 16 mm ($\frac{5}{8}$ in.) maximum.

13 Full Penetration Corner or Tee Joints *(1 July 2020)*

Measures taken to achieve full penetration corner or tee joints, where specified, are to be to the satisfaction of the attending Surveyor. Ultrasonic inspection of the member in way of the connection may be **called for** to assure the absence of injurious laminations prior to fabrication which could interfere with the attainment of a satisfactory welded joint.

CHAPTER 3 Environmental Conditions

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CHAPTER 3 Environmental Conditions

SECTION 1 Overview

1 General (1 July 2020)

Environmental conditions to which the Bottom-founded Offshore Wind Turbine is expected to be exposed during its design life are to be determined using adequate data for the areas in which the Support Structure is to be transported and installed. For the Support Structure requiring substantial near-shore construction (e.g., concrete gravity installations), environmental studies are to be appropriate with the duration of construction operations and the relative severity of expected conditions.

Environmental phenomena that influence the transport, installation, maintenance and operation of the Bottom-founded Offshore Wind Turbine are to be described in terms of relevant characteristic parameters. Statistical data and realistic statistical and mathematical models which describe the range of expected variations of environmental conditions are to be employed. All data used are to be fully documented with the sources and estimated reliability.

Methods employed in developing available data into design criteria are to be described and submitted in accordance with 1-1/9.1. Probabilistic methods for short-term, long-term and extreme-value prediction are to employ statistical distributions appropriate to environmental phenomena under consideration, as evidenced by relevant statistical tests, confidence limits and other measures of statistical significance. Hindcasting methods and models are to be fully documented if they are used to derive the environmental data.

Generally, suitable data and analyses supplied by consultants will be accepted as the basis for design. For installations in areas where published design standards and data exist, such standards and data may be cited in the design documentation.

3 Environmental Factors to be Considered (15 January 2013)

In general, the design of the Support Structure of an offshore wind turbine requires investigation of the following environmental factors.

- i) Wind
- ii) Waves
- iii) Currents
- iv) Tides, storm surges, and water levels
- v) Air and sea temperatures

- vi)* Air density
- vii)* Ice and snow accumulation
- viii)* Marine growth
- ix)* Seismicity
- x)* Sea ice or lake ice

Other phenomena, such as tsunamis, submarine slides, seiche, abnormal composition of air and water, air humidity, salinity, ice drift, icebergs, ice scouring, etc., may require investigation depending upon site-specific conditions and a specific design of the Support Structure.

The required investigation of seabed and soil conditions is described in 5-4/5.

CHAPTER 3 Environmental Conditions

SECTION 2 Wind

1 General

Statistical wind data are normally to include information on the frequency of occurrence, duration and direction of various wind speeds at the location where offshore wind turbines are to be installed. If on-site measurements are taken, the duration of individual measurements and the height above sea-level of measuring devices are to be stated. A wind speed value is only meaningful when qualified by its elevation and time-averaging duration.

In the absence of site data, published data and data from nearby land and sea stations may be used upon the agreement with ABS.

3 Wind Properties *(15 January 2013)*

3.1 Wind Speed and Turbulence

A wind condition is typically represented by a mean wind speed and a standard deviation of wind speed. The turbulence intensity, which measures the variation of wind speed relative to the mean wind speed, is defined as the ratio of the wind speed standard deviation to the mean wind speed (i.e. coefficient of variance of wind speed).

In this Guide, the mean wind speed, denoted as V_{hub} , at turbine hub height with 10-minute averaging duration is employed to define the turbulent wind conditions in the Design Load Cases (DLCs) in 4-2/3. The steady wind conditions as referred in the DLCs in 4-2/3 are defined by the mean wind speed with 1-minute or 3-second averaging duration. These turbulent and steady wind conditions are used to calculate the rotor generated aerodynamic loads as described in 4-3/3.1.

Mean wind speeds with 1-minute averaging time duration are used in the calculation of steady wind forces exerted on exposed components of the Support Structure, as described in 4-3/3.3.

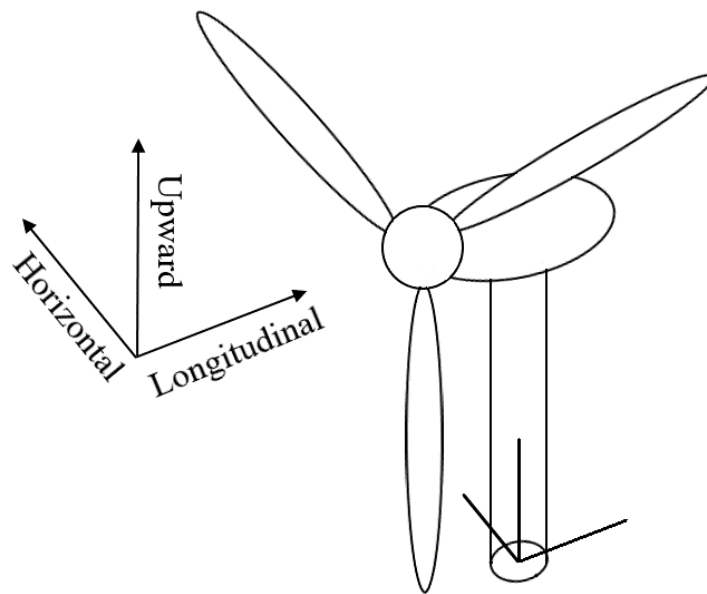
Conversion of a mean wind speed from one averaging duration to another is to be based on site-specific wind conditions. Use of conversion factors in recognized industry standards appropriate to the installation site may be acceptable and is subject to approval by ABS.

For wind speeds given in terms of the “fastest mile of wind”, V_f , the corresponding time-averaging period t in seconds is given by $t = 3600V_f$, where V_f is the fastest mile of wind at a reference height of 10 m (32.8 ft), in miles per hour.

The turbulence of wind over 10 minutes is generally considered stationary and can be modeled by a power spectral density function and a coherence function. The turbulence model is to include the effects of varying wind speed, shears and directions and allow rotational sampling through varying shears. The three vector components of turbulent wind velocity, as depicted in 3-2/3.1 FIGURE 1 are defined as:

- i) *Longitudinal* – Along the direction of the mean wind speed
- ii) *Lateral* – Horizontal and normal to the longitudinal direction
- iii) *Upward* – Normal to both the longitudinal and lateral directions and pointing upward

FIGURE 1
Vector Components of Turbulent Wind Velocity (1 July 2020)



3.3 Wind Profile (1 July 2020)

The mean wind speed profile (vertical wind shear) is to be defined by the power law:

$$V(z) = V_{hub}(z/z_{hub})^{\alpha}$$

where

- $V(z)$ = wind profile of the 10-minute mean wind speed as a function of height, z , above the SWL, in m/s (ft/s)
- V_{hub} = 10-minute mean wind speed at hub height, in m/s (ft/s)
- α = power law exponent, whose value is given in 3-2/9 and 3-2/11, (dimensionless)
- z = height above the SWL, in m (ft)
- z_{hub} = hub height above the SWL, in m (ft)

The power law wind profile can also be applied to the mean wind speeds with 1-minute or 1-hour averaging duration, except that the exponent is to be determined based on site-specific conditions or using published data.

For **extratropical** storm wind conditions, the mean wind speed profile may be represented by the following logarithmic wind shear law. It is expressed using the 1-hour mean wind speed at 10 m (32.8 ft) above the SWL.

$$V(z, t) = V(z, t_0)[1 - 0.41I_u(z)\ln(t/t_0)] \quad \text{for } t < t_0$$

where

$V(z, t)$	=	mean wind speed at height z and corresponding to an averaging time period t , in m/s (ft/s)
z	=	height above the SWL, in m (ft)
t	=	averaging time period shorter than $t_0 = 3600$ s, in seconds
$V(z, t_0)$	=	1-hour mean wind speed at height z , in m/s (ft/s)
	=	$V_o \left[1 + C \ln\left(\frac{z}{10\phi}\right) \right]$
t_0	=	reference averaging time period (1 hour), in seconds
	=	3600 s
C	=	$0.0573\sqrt{1 + 0.15V_o/\phi}$, (dimensionless)
V_o	=	1-hour mean wind speed at 10 m (32.8 ft) above the SWL, in m/s (ft/s)
$I_u(z)$	=	turbulence intensity (i.e., the ratio of the wind speed standard deviation to the mean wind speed) at height z , (dimensionless)
	=	$0.06[1 + 0.043V_o/\phi]\left(\frac{z}{10\phi}\right)^{-0.22}$
ϕ	=	unit conversion factor, (dimensionless)
	=	1 when using SI units (m, m/s)
	=	3.28 when using US Customary units (ft, ft/s)

For tropical cyclones (also termed as hurricanes or typhoons), the wind profile provided in Appendix 2 is to be applied.

For squalls, the wind profile recommended in API RP 2MET may be applied.

Other wind profile models may also be used provided that they can be justified by site-specific data.

3.5 Wind Spectrum and Spatial Coherence (1 July 2020)

Site-specific spectral density of wind speed and spatial coherence are to be determined based on measured wind data.

Unless site conditions indicate otherwise, the Kaimal spectrum and the exponential coherence model, as defined in A1-1/1, or the Mann uniform shear turbulence model, as recommended in Annex C of IEC 61400-1 (2019), is to be applied.

For the **extratropical storm** wind, API RP 2MET (2019) recommends using the NPD (Frøya) wind spectrum in conjunction with the two-point coherence function. Refer to A1-1/3 for the definitions.

For the tropical cyclone storm wind, the wind spectrum in A1-1/5 may be applied.

5 Long-Term and Extreme-Value Predictions

Long-term and extreme-value predictions for sustained and gust winds are to be based on recognized techniques and clearly described in the design documentation. Preferably, the statistical data used for the long-term distributions of wind speed are to be based on the same averaging periods of wind speeds as are used for the determination of loads.

7 Wind Conditions (1 July 2020)

A wind condition for the design of the Bottom-founded Offshore Wind Turbine is represented by a steady mean flow and an associated turbulence, as well as a mean wind direction and a change of the wind direction. The design wind conditions are further categorized into

- i) The normal wind conditions, which occur more frequently than once per year; and
- ii) The extreme wind conditions representing rare wind conditions with a given return period.

The normal and extreme wind conditions for the design of the Bottom-founded Offshore Wind Turbine are specified in 3-2/9 and 3-2/11. The design load conditions defined in Section 4-2 indicate how the wind condition is to be applied in combination with other environmental conditions.

9 Normal Wind Conditions

9.1 Normal Wind Profile Model (NWP) (15 January 2013)

The normal mean wind speed profile (vertical wind shear) is to be defined by the power law specified in 3-2/3.3, where the power law exponent $\alpha = 0.14$. The 10-minute mean wind speed at hub height, V_{hub} , is defined in 4-2/Table 1 and 4-2/Table 2.

9.3 Normal Turbulence Model (NTM) (1 July 2020)

The standard deviation of turbulence of the normal turbulence model, denoted as σ_{NTM} , is defined as the 90% quantile in the probability distribution of wind speed standard deviation conditioned upon a given 10-minute mean wind speed at hub height (V_{hub}).

The value of the turbulence standard deviation is to be determined using appropriate statistical techniques applied to measured and preferably de-trended data. Where the site assessment is not available, the recommended approach provided in Section 6.4.3 of IEC 61400-3-1 (2019) may be used to estimate the standard deviation as appropriate to the installation site.

The Normal Turbulence Model (NTM) is to be applied together with the Normal Wind Profile Model (NWP) as defined in 3-2/9.1.

11 Extreme Wind Conditions (1 July 2020)

(15 January 2013) The extreme wind conditions are represented by extreme wind shear events, peak wind speeds due to storms, extreme operating gust and turbulence, and rapid changes in wind speed and direction.

11.1 Extreme Wind Speed Model (EWM) (1 July 2020)

The Extreme Wind Speed Model (EWM) is defined as a turbulent wind model with a given return period as specified in 4-2/Table 1 and 4-2/Table 2 for the definition of Design Load Cases (DLCs).

For the EWM, when site data are not available, the wind profile of 10-minute mean wind speed is to be represented by the power law model as follows:

$$V(z) = V_{hub}(z/z_{hub})^{0.11}$$

where

$$\begin{aligned}
 V_{hub} &= 10\text{-minute mean wind speed at hub height, in m/s (ft/s)} \\
 &= V_{10\text{min}, n - \text{yr}} \\
 V_{10\text{min}, n - \text{yr}} &= 10\text{-minute mean wind speed at hub height with a return period of } n \text{ years, where } n \text{ is specified in 4-2/3 TABLE 1, in m/s (ft/s)} \\
 z &= \text{height above the SWL, in m (ft)} \\
 z_{hub} &= \text{hub height above the SWL, in m (ft)}
 \end{aligned}$$

The standard deviation of longitudinal turbulent wind speed of extreme wind condition, σ_1 , at hub height is to be calculated as:

$$\sigma_1 = 0.11 \times V_{hub}$$

For the extratropical storm wind, the logarithmic wind shear law given in 3-2/3.3 may be used to calculate the wind profile and standard deviation of extreme wind conditions.

For the tropical cyclone storm wind, the wind profile, the standard deviation, the turbulence intensity, and the gust factor provided in Appendix A2-1 are to be applied.

Where applicable, extreme squall events with 1-year and 50-year return periods in the time domain are to be considered as additional extreme wind conditions.

11.3 Extreme Operating Gust (EOG) (15 January 2013)

The Extreme Operating Gust (EOG) is represented by the hub height gust magnitude, V_{gust} , as defined in the following equation:

$$V_{gust} = \min \left\{ 1.35(V_{3\text{sec}, 1 - \text{yr}} - V_{hub}); \quad 3.3 \left(\frac{\sigma_{NTM}}{1 + 0.1D/\Lambda_1} \right) \right\}$$

where

$$\begin{aligned}
 V_{3\text{sec}, 1 - \text{yr}} &= 3\text{-second mean wind speed at hub height with a return period of 1 year, in m/s (ft/s)} \\
 V_{hub} &= 10\text{-minute mean wind speed at hub height, as defined in 4-2/3 TABLE 1, in m/s (ft/s)} \\
 \sigma_{NTM} &= \text{longitudinal turbulence standard deviation defined in 3-2/9.3, in m/s (ft/s)} \\
 \Lambda_1 &= \text{longitudinal turbulence length scale, in m (ft)} \\
 &= 0.7z \quad \text{when } z \leq 60 \text{ m (196.8 ft)} \\
 &= 42 \text{ m (137.8 ft)} \quad \text{when } z \geq 60 \text{ m (196.8 ft)} \\
 D &= \text{rotor diameter, in m (ft)}
 \end{aligned}$$

The time history of transient wind speed at height z is to be defined by:

$$V(z, t) = \begin{cases} V(z) - 0.37V_{gust} \sin(3\pi/T) [1 - \cos(2\pi/T)] & 0 \leq t \leq T \\ V(z) & \text{otherwise} \end{cases}$$

where

$V(z)$ = normal wind profile defined in 3-2/9.1, in m/s (ft/s)
 z = height above the SWL, in m (ft)
 T = 10.5 s

11.5 Extreme Turbulence Model (ETM)

The Extreme Turbulence Model (ETM) is to be represented by the Normal Wind Profile Model (NWP) model specified in 3-2/9.1 and the turbulence whose longitudinal component of the standard deviation is given by:

$$\sigma_1 = cI_{ref} \left[0.072 \left(\frac{V_{ave}}{c} + 3 \right) \left(\frac{V_{hub}}{c} - 4 \right) + 10 \right]$$

where

c = 2 m/s (6.56 ft/s)
 V_{hub} = 10-minute mean wind speed at hub height, as defined in 4-2/3 TABLE 1, in m/s (ft/s)
 V_{ave} = site-specific annual mean wind speed at hub height, in m/s (ft/s)
 I_{ref} = expected value of turbulence intensity at hub height when $V_{hub} = 15$ m/s (49.2 ft/s)

11.7 Extreme Direction Change (EDC) (15 January 2013)

The magnitude, θ , of the Extreme Direction Change (EDC) is to be calculated by:

$$\theta_e = \pm 4 \arctan \left(\frac{\sigma_{NTM}}{V_{hub}(1 + 0.1D/\Lambda_1)} \right) \quad -180^\circ \leq \theta_e \leq 180^\circ$$

where

σ_{NTM} = longitudinal turbulence standard deviation defined in 3-2/9.3, in m/s (ft/s)
 V_{hub} = 10-minute mean wind speed at hub height, as defined in 4-2/3 TABLE 1, in m/s (ft/s)
 Λ_1 = longitudinal turbulence length scale, defined in 3-2/11.3, in m (ft)
 D = rotor diameter, in m (ft)

The time history of transient extreme direction change, $\theta(t)$, is defined by:

$$\theta(t) = \begin{cases} 0^\circ & t < 0 \\ \pm 0.5\theta_e[1 - \cos(\pi t/T)] & 0 \leq t \leq T \\ \theta_e & t > T \end{cases}$$

where $T = 6$ s is the duration of the extreme direction change. The sign in the equation is to be chosen such that the most unfavorable transient loading occurs. At the end of the time history of direction change, the direction is assumed to remain a constant value (θ_e). The wind speed is to follow the normal wind profile (NWP) model in 3-2/9.1.

11.9 Extreme Coherent Gust with Direction Change (ECD)

The Extreme Coherent Gust with Direction Change (ECD) is to have a magnitude of:

$$V_{cg} = 15 \text{ m/s (49.21 ft/s)}$$

The time history of transient wind speed at height z , is defined by:

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

where

- $V(z)$ = normal wind profile defined in 3-2/9.1, in m/s (ft/s)
- T = rise time of gust wind, in seconds
= 10 s
- z = height above the SWL, in m (ft)

The rise in wind speed is assumed to occur simultaneously with the time history of direction change:

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

where

- θ_{cg} = magnitude of direction change, in degree
= $\begin{cases} 180^\circ & V_{hub} \leq 4\text{m/s (13.12ft/s)} \\ 720^\circ(V_{hub}/\phi)^{-1} & V_{hub} > 4\text{m/s (13.12ft/s)} \end{cases}$
- V_{hub} = 10-minute mean wind speed at hub height, in m/s (ft/s)
- T = rise time of gust wind, in seconds
= 10 s
- ϕ = unit conversion factor, (dimensionless)
= 1 when using SI units (m, m/s)
= 3.28 when using US Customary units (ft, ft/s)

11.11 Extreme Wind Shear (EWS) (1 July 2020)

The Extreme Wind Shear (EWS) is to be applied in both vertical and horizontal directions. The two extreme wind shears are considered independent events and therefore not to be applied simultaneously.

The time history of transient positive and negative vertical shear is given by:

$$V(z, t) = \begin{cases} V_{hub}\left(\frac{z}{z_{hub}}\right)^a \pm \left(\frac{z - z_{hub}}{D}\right)(2.5\phi + 0.2\beta\sigma_{NTM}(D/\Lambda_1)^{1/4})[1 - \cos(2\pi t/T)] & 0 \leq t \leq T \\ V_{hub}\left(\frac{z}{z_{hub}}\right)^a & \text{otherwise} \end{cases}$$

The time history of transient horizontal shear is given by:

$$V(z, t) = \begin{cases} V_{hub}\left(\frac{z}{z_{hub}}\right)^a \pm \left(\frac{y}{D}\right)(2.5\phi + 0.2\beta\sigma_{NTM}(D/\Lambda_1)^{1/4})[1 - \cos(2\pi t/T)] & 0 \leq t \leq T \\ V_{hub}\left(\frac{z}{z_{hub}}\right)^a & \text{otherwise} \end{cases}$$

where

V_{hub}	=	10-minute mean wind speed at hub height, as defined in 4-2/3 TABLE 1, in m/s (ft/s)
σ_{NTM}	=	longitudinal turbulence standard deviation defined in 3-2/9.3, in m/s (ft/s)
Λ_1	=	longitudinal turbulence length scale, defined in 3-2/11.3, in m (ft)
z	=	height above the SWL, in m (ft)
z_{hub}	=	hub height above the SWL, in m (ft)
y	=	horizontal distance from hub in the cross wind direction, in m (ft)
D	=	rotor diameter, in m (ft)
α	=	0.14, (dimensionless)
β	=	6.4, (dimensionless)
T	=	12 s
ϕ	=	unit conversion factor, (dimensionless)
	=	1 when using SI units (m, m/s)
	=	3.28 when using US Customary units (ft, ft/s)

The sign for the transient wind shear is to be determined such that the most unfavorable transient loading occurs.

13 Survival Wind Conditions (1 July 2020)

The survival wind conditions are described by the Survival Wind Speed Model (SurWM) for the robustness assessment for the Bottom-Funded Offshore Wind Turbines in tropical cyclone-prone areas.

13.1 Survival Wind Speed Model (SurWM)

The Survival Wind Speed Model (SurWM) is similar to the Extreme Wind Speed Model (EWM) defined in 3-2/11.1, but with a return period longer than 50 years, as specified in 4-2/5.

When site data are not available, the wind profile and turbulence spectrum for the SurWM are to be defined according to 3-2/3.3 and 3-2/3.5.

CHAPTER 3 Environmental Conditions

SECTION 3 Waves

1 General (1 July 2020)

The development of wave data is to reflect conditions at the installation site and the type of the Support Structure. Statistical wave data from which design parameters are determined are normally to include the frequency of occurrence of various wave height groups, associated wave periods and directions. Published data and previously established design criteria for **specific** areas may be used where such exist. Hindcasting techniques that adequately account for the shoaling and fetch limited effects on wave conditions at the site may be used to augment available data. Analytical wave spectra employed to augment available data are to reflect the shape and width of the data and are to be appropriate to the general site conditions.

As applicable, wave data are to be developed to determine the following:

- i)* Provision for air gap
- ii)* Maximum mud line shear force and overturning moment
- iii)* Dynamic response of the Support Structure
- iv)* Maximum stress
- v)* Fatigue
- vi)* Wave impact on the local structure

All long-term and extreme value predictions employed for the determination of design wave conditions are to be fully described and based on recognized techniques. Because the wave-induced responses may be increased due to the change of wave period and direction, consideration is to be given to waves of less than the maximum height but with a different period and/or direction. Waves that cause the most unfavorable effects on the overall structure may also differ from waves having the most severe effects on individual structural components. In addition to the most severe wave conditions, frequent waves of smaller heights are to be investigated to assess their effect on the fatigue and dynamic responses.

The wave conditions for the design of the Bottom-founded Offshore Wind Turbine are described in **3-3/3 through 3-3/11**. The design load conditions defined in Chapter 4, Section 2 specify how these wave conditions are to be applied in combination with other design environmental conditions.

3 Normal Sea State (NSS) (1 July 2020)

The Normal Sea State (NSS) is a stochastic design sea state represented by a significant wave height, a peak spectral period, and a wave direction. It is to be determined based on the site-specific long-term joint probability distribution of metocean parameters conditioned upon a given 10-minute mean wind speed at hub height, V_{hub} .

The Normal Sea State (NSS) is used in Chapter 4, Section 2 to define the Design Load Cases (DLCs) requiring either strength analysis or fatigue analysis.

For strength analyses, the Normal Sea State (NSS) can be characterized by the expected value of significant wave height, $H_{s,NSS}$, conditioned upon a given value of V_{hub} (i.e., $H_{s,NSS} = E[H_s | V_{hub}]$). A range of peak period, T_p , associated with each significant wave height is to be determined for load calculations. The resultant most unfavorable responses are to be used in the design of the Support Structure.

For fatigue analyses, the number and resolution of the Normal Sea State (NSS) are to be determined in such a manner that the fatigue damage associated with the full long-term distribution of metocean parameters can be sufficiently accounted for.

5 Severe Sea State (SSS) (1 July 2020)

The Severe Sea State (SSS) is a stochastic design sea state to be applied in combination with the normal wind conditions as specified in 3-2/9 for the strength analysis of the Support Structure when the turbine is in the power production mode.

The Severe Sea State (SSS) is represented by a significant wave height, $H_{s,SSS}(n\text{-yr})$, a peak spectral period and a wave direction. It is to be determined by extrapolation of site-specific long term joint probability distribution of metocean parameters to the extent that the joint occurrence of $H_{s,SSS}(n\text{-yr})$ and a given value of 10-minute mean wind speed, V_{hub} , at hub height has a specified return period of n years, where n is defined in 4-2/3 TABLE 1. A series of V_{hub} is to be selected within the range of mean wind speed corresponding to power production. As a conservative estimation, the 100-year return significant wave height independent of wind speed may be used to approximate $H_{s,SSS}(n\text{-yr})$.

A range of peak period associated with each significant wave height is to be determined for the load calculation. The resultant most unfavorable responses are to be used in the design of the Support Structure.

7 Extreme Sea State (ESS) (1 July 2020)

The Extreme Sea State (ESS) is a stochastic extreme design sea state with a given return period as defined in 4-2/3 TABLE 1 for specific design load cases.

The Extreme Sea State (ESS) is represented by a significant wave height, $H_{s,n-yr}$, with a return period of n years, a peak spectral period and a wave direction. The values of $H_{s,n-yr}$ are to be determined from on-site measurements, hindcast data, or both for the installation site. A range of peak spectral periods appropriate to site-specific metocean conditions is to be determined for load calculations, and the resultant most unfavorable responses are to be used in the design of the Support Structure.

9 Survival Sea State (SurSS) (1 July 2020)

The Survival Sea State (SurSS) condition is similar to the Extreme Sea State (ESS) defined in 3-3/7, but with a return period longer than 50 years, as specified in 4-2/5.

11 Breaking Waves (1 July 2020)

Where breaking waves are likely to occur at the installation site, the loads exerted by those breaking waves are to be assessed in the design. Breaking wave criteria are to be appropriate to the installation site and based on recognized methods. In shallow water the empirical limit of the wave height is approximately

0.78 times the local water depth. In deep water, a theoretical limit of wave steepness prior to breaking is $1/7$.

Further guidance on breaking wave hydrodynamics can be found in IEC 61400-3-1 (2019), Annex B.

CHAPTER 3 Environmental Conditions

SECTION 4 Currents

1 Currents (1 July 2020)

Data for currents are to include information on current speed, direction, and variation with depth. The extent of information needed is to be commensurate with the expected severity of current conditions at the site in relation to other load-causing phenomena, past experience in adjacent or **similar** areas, and the type of structure and foundation to be installed. On-site data collection may be required for previously unstudied areas or areas expected to have unusual or severe conditions. Consideration is to be given to the following types of current, as appropriate to the installation site:

Wind-generated current

Tide, density, circulation, and river-outflow generated sub-surface current

Near shore, breaking wave induced surface currents running parallel to the coast

The direction of wind generated surface current velocity is assumed to be aligned with the wind direction.

For near shore, the breaking wave induced surface current runs in parallel to the coastline, **current** speed may be estimated by the following equation:

$$U_b = 2\zeta\sqrt{gH_b}$$

where

U_b = speed of the near shore, breaking wave induced surface current running parallel to the coastline, in m/s (ft/s)

ζ = sea floor slope, in rad

g = acceleration of gravity, in m/s² (ft/s²)

H_b = breaking wave height, in m (ft)

Current velocity profiles with depth are to be based on site-specific data or recognized empirical relationships. Unusual profiles due to bottom currents and stratified effects due to river out-flow currents are to be accounted for. For the design of offshore wind turbines to be installed on the US Outer Continental Shelf, the current profile is to be determined in accordance with **Annex H, Annex I and Annex J of API RP 2MET (2019)**.

The current models for the design of the Bottom-founded Offshore Wind Turbine are described in 3-4/3 to 3-4/7. The design load conditions defined in Section 4-2 specify how the current models are to be applied in combination with other design environmental conditions.

3 Normal Current Model (NCM) (15 January 2013)

The Normal Current Model (NCM) is to be determined based on the long-term joint probability distribution of metocean parameters at the installation site.

For strength analyses, the Normal Current Model (NCM) is to represent the site-specific wind generated current conditioned upon a given 10-minute mean wind speed at hub height (i.e., V_{hub}). Tide and storm-generated sub-surface currents are not included.

For fatigue analyses, the Normal Current Model (NCM) is to be determined in such a manner that the fatigue damage associated with the full long-term distribution of metocean parameters can be sufficiently accounted for.

5 Extreme Current Model (ECM) (15 January 2013)

The extreme current model is defined as site-specific currents with a given return period as specified in 4-2/3 TABLE 1.

The Extreme Current Model (ECM) is to be applied in combination with the Extreme Wind Model (EWM) defined in 3-2/11.1, the Extreme Sea State (ESS) defined in 3-3/7, and the Extreme Water Level Range Model (EWLR) defined in 3-5/5, with due consideration of their joint occurrence probabilities as required in 4-2/3.x

7 Survival Current Model (SurCM) (1 July 2020)

The Survival Current Model (SurCM) is similar to the Extreme Current Model (ECM) defined in 3-4/5, but with a return period longer than 50 years, as specified in Section 4-2.

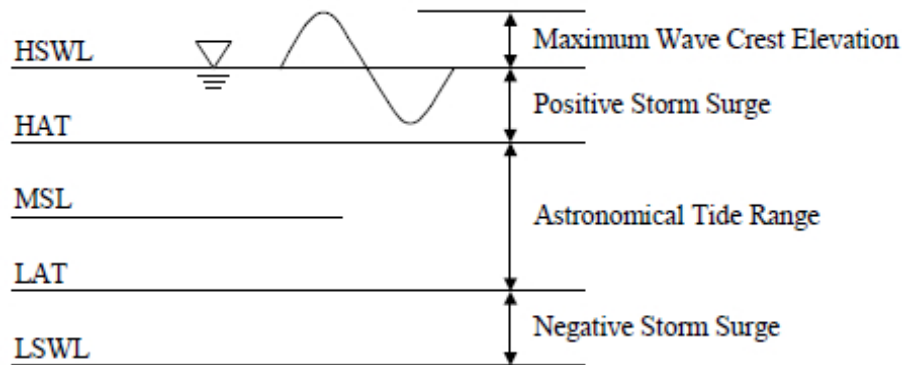
CHAPTER 3 Environmental Conditions

SECTION 5 Tides, Storm Surges, and Water Levels

1 General (1 July 2020)

Tides can be classified as lunar or astronomical tides, wind tides, and pressure differential tides. The combination of the latter two is commonly called the storm surge. The water depth at any location consists of the mean depth, defined as the vertical distance between the sea floor and an appropriate near-surface datum, and a fluctuating component due to astronomical tides and storm surges. Astronomical tide variations are bounded by the highest astronomical tide (HAT) and the lowest astronomical tide (LAT). Storm surge is to be estimated from available statistics or by mathematical storm surge modeling. The still water level (SWL) referred in the definition of environmental conditions for load calculation is to be taken as the highest still water level (HSWL), which is defined as the sum of the highest astronomical level and the positive storm surge. Definitions of various water levels referred in this Guide are illustrated in 3-5/1 FIGURE 1.

FIGURE 1
Definitions of Water Levels



HSWL	Highest Still Water Level
HAT	Highest Astronomical Tide
MSL	Mean Sea Level (Mean Still Water Level)
LAT	Lowest Astronomical Tide
LSWL	Lowest Still Water Level

Variations in the elevation of the daily tide may be used in determining the elevations of boat landings, barge fenders and the top of the splash zone for corrosion protection of structure. Water depths assumed for various types of analysis are to be clearly stated.

The water level ranges for the design of the Bottom-founded Offshore Wind Turbine are defined in 3-5/3 to 3-5/9. The design load conditions defined in Section 4-2 specify how the water level ranges are to be applied in combination with other design environmental conditions.

3 Normal Water Level Range (NWLR) (15 January 2013)

The Normal Water Level Range (NWLR) is defined as the variation in water level with a return period of one year. In the absence of site-specific long-term probability distribution of water levels, the normal water level range may be approximated by the variation between highest astronomical tide (HAT) and lowest astronomical tide (LAT).

Load calculations for the strength analysis are to be performed based on the water level within the Normal Water Level Range (NWLR) in order to determine the most unfavorable responses. The influence of water level variation on fatigue loads is also to be considered, if deemed necessary.

5 Extreme Water Level Range (EWLR) (1 July 2020)

The Extreme Water Level Range (EWLR) is to be assumed for load cases associated with extreme wave conditions with a given return period as specified in 4-2/3 TABLE 1. Load calculations for the strength analysis are to be performed based on the water level within the Extreme Water Level Range (EWLR) to determine the most unfavorable responses of the Support Structure.

In the absence of the long-term joint probability distribution of metocean parameters including water level, the following water levels are to be considered as a minimum:

- The highest still water level (HSWL), defined as a combination of highest astronomical tide (HAT) and positive storm surge, with a given return period as specified in 4-2/3 TABLE 1

- The lowest still water level (LSWL), defined as a combination of lowest astronomical tide (LAT) and negative storm surge, with a given return period as specified in 4-2/3 TABLE 1
- The water level associated with the highest breaking wave load

7 Survival Water Level Range (SurWLR) (1 July 2020)

The Survival Water Level Range (SurWLR) is defined in a manner similar to the Extreme Water Level Range (EWLR) in 3-5/5, but with a return period longer than 50 years, as specified in Section 4-2.

CHAPTER 3 Environmental Conditions

SECTION 6 Other Conditions

1 **Temperature** *(1 July 2020)*

Extreme values of air, sea and seabed temperatures are to be expressed in terms of return periods and associated highest and lowest values. Wind speed data are typically presented with respect to a specific reference temperature. Temperature data is also to be used to evaluate the selection of air density, structural materials, ambient ranges and conditions for machinery and equipment design, and for determination of thermal stresses, as relevant to the Bottom-founded Offshore Wind Turbine.

3 **Air Density** *(15 January 2013)*

The air density is to be measured in conjunction with the wind conditions at the installation site.

Where there are no site data for the air density, the value of air density is to be determined according to ISO 2533 and corrected as appropriate for annual average temperature at the installation site.

5 **Ice and Snow Accumulation**

For an installation site where ice and snow may accumulate, estimates are to be made of the extent to which ice and snow may accumulate. Data are to be derived from actual field measurements, laboratory data or data from analogous areas.

7 **Marine Growth** *(1 July 2020)*

Marine growth is to be considered in the design of the Bottom-founded Offshore Wind Turbine. Estimates of the rate and extent of marine growth may be based on past experience and available field data. Particular attention is to be paid to increases in hydrodynamic loading due to increased diameters and surface roughness of members caused by marine fouling as well as to the added weight and increased inertial mass of submerged structural members. The types of fouling likely to occur and their possible effects on corrosion protection coatings are to be considered.

9 **Seismicity and Earthquake Related Phenomena** *(1 July 2020)*

The effects of earthquakes on the Bottom-founded Offshore Wind Turbine located in areas known to be seismically active are to be taken into account.

9.1 Levels of Earthquake Conditions (1 July 2020)

The magnitudes of the parameters characterizing the earthquakes with return periods appropriate to the design life and the intended safety level of the Bottom-founded Offshore Wind Turbine are to be determined. Two levels of earthquake conditions are to be considered to address the risk of damage and structure collapse, respectively:

- i) *Strength Level.* Ground motion which has a reasonable likelihood of not being exceeded at the site during the design life of the Bottom-founded Offshore Wind Turbine
- ii) *Ductility Level.* Ground motion for a rare, intense earthquake to be applied to evaluate the risk of structural collapse.

9.3 Regional and Site-specific Data (1 July 2020)

The anticipated seismicity of an area is, to the extent practicable, to be established based on suitable regional and **site-specific** data including the following:

- i) Magnitudes and recurrence intervals of seismic events
- ii) Proximity to active faults
- iii) Type of faulting
- iv) Attenuation of ground motion between the faults and the site
- v) Subsurface soil conditions
- vi) Records from past seismic events at the site where available, or from analogous sites

9.5 Other Earthquake Related Phenomena (1 July 2020)

Seismic data are to be used to establish quantitative Strength Level and Ductility Level earthquake criteria describing the earthquake induced ground motion expected during the design life of the Bottom-founded Offshore Wind Turbine. In addition to ground motion, and as applicable to the site in question, the following earthquake related phenomena are to be taken into account.

- i) Liquefaction of subsurface soils submarine slides
- ii) Tsunamis
- iii) Acoustic overpressure shock waves

11 Sea Ice or Lake Ice

For an installation site where ice hazards may occur, the effects of sea ice or lake ice on the Support Structure of an offshore wind turbine are to be taken into account in the design. Depending on the ice conditions at the site, the Support Structure may encounter moving ice and fast ice cover.

Statistical ice data of the site are to be used as the base for deriving the parameters such as ice thickness, ice crushing strength and pack ice concentration, etc., which are required for determining the ice loads.

Impact, both centric and eccentric, is to be considered where moving ice may impact the Support Structure. Impact of smaller ice masses, which are accelerated by storm waves, and of large masses (multi-year floes and icebergs) moving under the action of current, wind, and Coriolis effect is to be considered in the design.

The interaction between ice and the Support Structure produces responses both in the ice and the structure-soil system, and this compliance is to be taken into account as applicable.

13 Lightning *(1 July 2020)*

Lightning protection is to be designed in accordance with IEC 61400-24. It is not necessary for protective measures to extend to all parts of the Bottom-founded Offshore Wind Turbine, provided that safety is not compromised.

15 Electric Network Conditions *(1 July 2020)*

The assessment of electric network conditions is to be performed in accordance with IEC 61400-3-1 (2019).

CHAPTER 4 Loads

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CHAPTER 4 Loads

SECTION 1 Overview

1 General (1 July 2020)

This Section pertains to the identification, definition and determination of the loads which the Bottom-founded Offshore Wind Turbine may experience in its pre-service (load-out, transportation, installation, and commissioning) and in-service (operation, maintenance, and repair) phases. As appropriate to the planned Bottom-founded Offshore Wind Turbine, the types of loads described in 4-1/3 are to be considered in the design.

3 Types of Loads to be Considered (1 July 2020)

Loads applied to the Bottom-founded Offshore Wind Turbine are, for the purposes of this Guide, categorized as dead loads, live loads, deformation loads and environmental loads.

3.1 Dead Loads (1 July 2020)

Dead loads are loads which do not change during the mode of operation under consideration. Dead loads include, but are not limited to, the following:

- i) Weight of rotor components (blades, hub, shaft, etc.) and equipment inside the nacelle (control and protection components, gearbox, drive train components, electrical generation components, cables, etc.)
- ii) Weight of nacelle housing structure, tower, substructure, foundation, access platform, fenders, ladders, corrosion protection system, and other permanent structures.
- iii) External hydrostatic pressure and buoyancy calculated **based on** the SWL

Static earth pressure

3.3 Live Loads (1 July 2020)

Live loads associated with the normal operation of a wind turbine are loads which may change during the mode of operation considered. Live loads acting after construction and installation may include, but are not limited to, the following.

- i) Forces exerted on the wind turbine due to lifting equipment during installation and maintenance of the wind turbine.
- ii) Forces exerted by vessels moored to the structure or routine impact loads from a typical supply vessel that would normally service the offshore wind turbine

- iii) Loads associated with helicopter operations, where relevant.
- iv) Actuation loads generated by wind turbine operations and controls including torque control from a generator or inverter, yaw and pitch actuator loads, and mechanical braking loads. The range of actuator forces is to be considered as appropriate in the calculation of response and loading. In particular, the range of friction, spring force or pressure for mechanical brakes is influenced by temperature and ageing, which are to be taken into account when calculating the response and loading during any brake event.

Where applicable, the dynamic effects of the live loads on the Support Structure are to be taken into account.

Live loads encountered during load-out, transportation and installation are to be determined for each specific operation involved. The dynamic effects of such loads, as necessary, are to be accounted for (see [Section 5-5](#)).

3.5 Deformation Loads

Deformation loads are loads due to deformations imposed on the Support Structure. The deformation loads include those due to temperature variations leading to thermal stress in the structure and, where necessary, loads due to soil displacements (e.g., differential settlement or lateral displacement) or due to deformations of adjacent structures. For concrete structures, deformation loads due to prestress, creep, shrinkage and expansion are to be taken into account.

3.7 Environmental Loads (1 July 2020)

Environmental loads are loads due to the action of wind, wave, current, ice, snow, earthquake, marine growth and other environmental phenomena as described in Chapter 3. The characteristic parameters defining environmental loads are to be appropriate to the installation site and in accordance with the requirements specified in Chapter 3 of this Guide. The combination and severity of environmental conditions for the design of the Support Structure are specified in Section 4-2. Calculations of environmental loads are to be in accordance with Section 4-3.

Environmental loads are to be applied from directions producing the most unfavorable effects on the Support Structure unless site-specific studies provide evidence in support of a less stringent requirement. Directionality may be taken into account in applying the environmental criteria.

CHAPTER 4 Loads

SECTION 2 Design Load Conditions

1 General (1 July 2020)

Design load conditions of the Bottom-founded Offshore Wind Turbine are to be represented by a set of Design Load Cases (DLCs), which are defined by various combinations of turbine operational modes, site-specific environmental conditions, electrical network conditions and other applicable design conditions, such as specific transportation, assembly, maintenance or repair conditions. All relevant DLCs with a reasonable probability of occurrence and covering the most significant conditions that an offshore wind turbine may experience are to be considered in the design.

For the Bottom-founded Offshore Wind Turbine installed in a tropical cyclone region, Survival Load Cases (SLCs) may be assessed to verify its structural robustness.

Assessment of the tower for the design load conditions defined in this Chapter is required for the following situations (see 1-1/3):

- The optional **RNA** notation is requested by the Owner
- The optional **RNA** notation is not requested by the Owner, however the tower and its connection to the RNA and/or to the substructure are not included in the wind turbine type certificate

3 Definition of Design Load Cases (DLCs) (1 July 2020)

As a minimum, the DLCs defined in 4-2/Table 1 and 4-2/Table 2 are to be suitably assessed in the design of the Bottom-founded Offshore Wind Turbine. Other design load cases are to be considered, wherever they are deemed relevant to the structural integrity of a specific design of the Support Structure. In particular, if correlation exists between an extreme environmental condition and a fault condition of wind turbine, a realistic combination of the two is to be considered as a design load case.

For each DLC defined in 4-2/Table 1 and 4-2/Table 2, the 'Type of Analysis' is denoted 'S' for the strength analysis or 'F' for the fatigue analysis. Results of the strength analysis are used in the structural assessment against acceptance criteria pertaining to the yielding and buckling. Results of the fatigue analysis are used in the structural assessment against criteria pertaining to fatigue performance.

Those DLCs indicated with 'S', are further classified as

- i) 'N' representing normal design conditions, which are expected to occur frequently during the design life of the Bottom-founded Offshore Wind Turbine. The corresponding operational mode of the turbine is in a normal state or with minor faults or abnormalities.

- ii) ‘A’ representing abnormal design conditions, which are less likely to occur than normal design conditions. They usually correspond to design conditions with severe faults that result in activation of system protection functions.
- iii) ‘T’ representing the design conditions relevant to temporary operations including load-out, transportation, installation, maintenance and repair.

The type of design conditions, ‘N’, ‘A’, or ‘T’ determines the safety factors in the strength design criteria to be applied to the Support Structure. These safety factors are defined in Chapter 5.

The DLCs specified in 4-2/Table 1 are generally in agreement with those required by “Table 2 – Design load cases” of IEC 61400-3-1 (2019), with various modifications to:

- i) Address the design requirements for the Support Structure to be installed in an offshore site where the extreme environmental condition is governed by tropical cyclones, (also known as hurricanes or typhoons); and
- ii) Provide further interpretations of the DLCs in terms of site-specific design requirements for the Support Structure.

The following statements pertain to the required DLCs in 4-2/3 TABLE 1. Further guidance on the analysis requirements can be found in Section 7.4 and Section 7.5.4 of IEC 61400-3-1 (2019).

- i) The DLCs defined in 4-2/3 TABLE 1 are for the design of the Bottom-founded Offshore Wind Turbine, which represents the site-specific Support Structure of an offshore wind turbine. As such, DLC 1.1 required by IEC 61400-3-1 (2019) for the calculation of ultimate loads acting on the RNA is not included in 4-2/3 TABLE 1.
- ii) The safety factors indicated in the column entitled ‘Safety Factor’ in 4-2/3 TABLE 1 are specified in Chapter 5 of this Guide.
- iii) The design environmental conditions referred in 4-2/3 TABLE 1 for wind, waves, sea currents, and water level ranges are in accordance with the definitions specified in Chapter 3 of this Guide. Detailed references are listed in the table notes.
- iv) Site-specific extreme wind speeds with various combinations of return periods and averaging time durations are used to define the environmental conditions in DLC’s with serial number 6.1, 6.2, 6.3, 6.4, 7.1, 7.2, 8.2 and 8.3. This differs from IEC 61400-3-1 (2019) where the extreme wind speeds in these DLCs are defined using the RNA’s Reference Wind Speed (V_{ref}) in combination with various prescribed conversion factors for different averaging time durations or return periods.
- v) The return period n of each extreme or severe environmental condition is specified in the column entitled ‘Other Conditions’ in 4-2/Table 1.

Any reduction to the return period is subject to special consideration by ABS.

The return period is generally not to be less than 50 years for the extreme environmental conditions of DLC 6.1 and DLC 6.2 and the severe wave conditions of DLC 1.6.

Such return period is to be increased to 100 years for designing the Support Structure of an offshore wind turbine at an installation site where the extreme storm condition is expected to be determined by tropical cyclones (i.e., tropical cyclone-prone site, as indicated in 4-2/3 TABLE 1). The increase in the return period is to address higher variations of the severity of extreme storm in tropical cyclone-prone offshore regions such as the Gulf of Mexico and the East Atlantic Coast in the U.S., where the extreme wind speed versus the return period curve typically shows a steep slope.

The reduction of the return period of 100 years to 50 years in tropical cyclone prone areas may be acceptable provided additional safety factors are applied or robustness assessments for the Survival Load Cases (see 4-2/5) are performed to justify such reduction.

- vi) DLC 6.2 assume a loss of connection to electrical power network at an early stage of the storm containing the extreme wind condition. A nacelle yaw misalignment ranging between -180° and $+180^\circ$ is generally required to be considered. Load calculations are to be based on the misalignment angle that results in the most unfavorable responses in the Bottom-founded Offshore Wind Turbine. The range of yaw misalignment may be reduced to account for the contribution from an active or passive yaw control system, provided the designer can justify that
- Such a system is capable of achieving the assumed reduction of yaw misalignment under site-specific conditions; and
 - An appropriate monitoring and maintenance program is implemented to maintain the effectiveness of yawing control during the design life of the Bottom-founded Offshore Wind Turbine.
- vii) For a site where the turbulent wind models defined in Chapter 3, Section 2 are considered insufficient to describe site-specific wind conditions, the DLC's in 4-2/3 TABLE 1 involving the combined steady wind model and deterministic design wave height are to be evaluated for the strength analysis.
- viii) For those load cases, including DLC 1.6, DLC 6.1, 6.2, 6.3, 7.1, and 8.2, which in general require time domain dynamic simulations for the combined turbulent wind and stochastic extreme waves, the simulation time duration may differ from the reference periods of wind speed and significant wave height. The extreme responses determined for the wave condition used in the simulation are to be equivalent to those obtained from 3-hour simulations.
- IEC 61400-3-1 (2019), Section 7.5.6 recommends at least six 1-hour constrained wave simulations be performed. Other approaches may be taken if the estimated extreme response is shown not less than that obtained with the approach recommended by IEC 61400-3-1 (2019), Section 7.5.6.
- ix) Where a wind speed range is indicated in 4-2/3 TABLE 1, wind speeds leading to the most unfavorable responses are to be considered for the structural and foundation design. When the range of wind speeds is represented by a set of discrete values, the interval between two adjacent discrete wind speeds is not to be greater than 2 m/s (66 ft/s). In addition, the turbine Rated Wind Speed (V_r , see 1-4/3.39), where applicable, is to be included as one of the discrete wind speeds to be used in the load calculation.
- x) Extreme metocean conditions in DLC 6.1, 6.2, 6.3, 7.1 and 8.2 are formed by combining the turbulent extreme wind (EWM), the stochastic extreme wave (ESS), the extreme current (ECM), and the extreme water level range (EWLR). The probability of joint occurrence of these environmental parameters is to be taken into account when establishing extreme metocean conditions. Consideration is to be given to the peak wind, peak wave and peak current condition (see e.g., API RP 2MET, 2019), as appropriate to site conditions and a specific design of the Support Structure. Combining all individual extremes with the same return period is normally a conservative approach.
- xi) For those DLCs denoted by 'S' in the 'Type of Analysis' column in 4-2/3 TABLE 1 for the structural strength design, the effect of environmental loads are to be suitably combined with the effect of dead loads, live loads, and deformation loads (see 4-1/3). Combinations of the load effects that produce the most severe local and/or global effects on the Support Structure are to be considered.
- xii) If site-specific directional data are not available, the direction of applied environmental conditions is to be determined to produce the most unfavorable effect on the Support Structure. For DLCs with the serial number 6.1, 6.2, 6.3, 7.1 and 8.3, the misalignment between wind and wave directions is to be considered up to 90° for extreme environmental conditions governed by tropic cyclones.
- xiii) The effect of currents is required to be considered in those DLCs denoted by 'F' (i.e., fatigue analysis) in the 'Type of Analysis' column in 4-2/3 TABLE 1.

- xiv)* A new DLC 8.4 is added to account for the fatigue damage occurred during transport, assembly, maintenance and repair.
- xv)* Wind and wave directionality for DLC 1.2, 1.6, 6.4, 7.2 and 8.3 is to consider the misalignment of wind and wave directions. If site-specific directional data are not available, the direction of applied environmental conditions is to be determined to produce the most unfavorable effect on the Support Structure.

For the Bottom-founded Offshore Wind Turbine to be installed at a site where ice is expected to occur, the DLCs specified in 4-2/3 TABLE 2 for ice conditions are to be considered.

The DLCs specified in 4-2/3 TABLE 2 for ice conditions are generally in agreement with those required by “Table 3 – Design load cases for sea ice” in IEC 61400-3-1 (2019). The descriptions and analysis requirements for DLCs defined in 4-2/3 TABLE 2 can be referred to Annex D of IEC 61400-3-1 (2019), in conjunction with the amendments described as follows:

- i)* The safety factors referred in 4-2/3 TABLE 2 are defined in Chapter 5 of this Guide.
- ii)* The design environmental conditions referred in 4-2/3 TABLE 2 for ice, wind and water level ranges are in accordance with the definitions in Chapter 3 of this Guide
- iii)* Site-specific extreme wind speeds with a given return period and averaging time duration are used to define the extreme wind conditions.
- iv)* The return period chosen for the extreme ice conditions of DLC D3, D4 and D7 is generally not to be less than 50 years. Any reduction to the return period of environmental conditions is subject to special consideration by ABS.
- v)* Reference is also made to API RP 2N and ISO 19906 for the alternative methods of determining static and dynamic ice loads.

TABLE 1
Design Load Cases (1 July 2020)

<i>Design Condition</i>	<i>DLC</i>	<i>Wind Condition</i>	<i>Waves</i>	<i>Wind and Wave Directionality</i>	<i>Sea Currents</i>	<i>Water Level</i>	<i>Other Conditions</i>	<i>Type of Analysis</i>	<i>Safety Factor</i>
1) Power production	1.2	NTM $V_{in} \leq V_{hub} \leq V_{out}$	NSS Joint prob. distribution of H_s, T_p, V_{hub}	MIS, MUL	NCM	NWLR or \geq MSL		F	FDF
	1.3	ETM $V_{in} \leq V_{hub} \leq V_{out}$	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	MSL		S	N
	1.4	ECD $V_{hub} = V_r - 2\text{m/s}(6.6\text{ft/s}),$ $V_r, V_r + 2\text{m/s}(6.6\text{ft/s})$	NSS $H_s = E[H_s V_{hub}]$	MIS, wind direction change	NCM	MSL		S	N
	1.5	EWS $V_{in} \leq V_{hub} \leq V_{out}$	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	MSL		S	N
	1.6	NTM $V_{in} \leq V_{hub} \leq V_{out}$	SSS $H_s = H_{SSSS} (n - \text{yr})$	MIS, MUL	NCM	NWLR	$n = 50$ in general; $n = 100$ for tropical cyclone-prone sites (See Note 1 for further guidance)	S	N

Design Condition	DLC	Wind Condition	Waves	Wind and Wave Directionality	Sea Currents	Water Level	Other Conditions	Type of Analysis	Safety Factor
2) Power production plus occurrence of fault	2.1	NTM $V_{in} \leq V_{hub} \leq V_{out}$	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	MSL	Normal control system fault or loss of electrical network or primary layer control function fault	S	N
	2.2	NTM $V_{in} \leq V_{hub} \leq V_{out}$	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	MSL	Abnormal control system fault or secondary layer protection function related fault	S	A
	2.3	EOG $V_{hub} = V_r \pm 2m/s (6.6ft/s)$ and V_{out}	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	MSL	External or internal electrical fault including loss of electrical network	S	A
3) Start-up	2.4	NTM $V_{in} \leq V_{hub} \leq V_{out}$	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	NWLR or \geq MSL	Control system fault, electrical fault or loss of electrical network	F	FDF
	2.5	NTM $V_{in} \leq V_{hub} \leq V_{out}$	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	MSL	Low voltage ride through	S	N
	3.1	NWP $V_{in} \leq V_{hub} \leq V_{out}$	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	NWLR or \geq MSL		F	FDF
	3.2	EOG $V_{hub} = V_{in}, V_r \pm 2m/s (6.6ft/s)$ and V_{out}	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	MSL		S	N
	3.3	EDC $V_{hub} = V_{in}, V_r \pm 2m/s (6.6ft/s)$ and V_{out}	NSS $H_s = E[H_s V_{hub}]$	MIS, wind direction change	NCM	MSL		S	N

Design Condition	DLC	Wind Condition	Waves	Wind and Wave Directionality	Sea Currents	Water Level	Other Conditions	Type of Analysis	Safety Factor
4) Normal shut down	4.1	NWP $V_{in} \leq V_{hub} \leq V_{out}$	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	NWLR or \geq MSL		F	FDF
	4.2	EOG $V_{hub} = V_r \pm 2m/s (6.6ft/s)$ and V_{out}	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	MSL		S	N
5) Emergency stop	5.1	NTM $V_{hub} = V_r \pm 2m/s (6.6ft/s)$ and V_{out}	NSS $H_s = E[H_s V_{hub}]$	COD, UNI	NCM	MSL		S	N
6) Parked (standing still or idling)	6.1	EWM Turbulent wind model $V_{hub} = V_{10min, n-yr}$	ESS $H_s = H_{s, n-yr}$	MIS, MUL	ECM $n-yr$ Current	EWLR $n-yr$ Water Level	$n = 50$ in general; $n = 100$ for tropical cyclone-prone sites (See Note 1 for further guidance.)	S	N
	6.2	EWM Turbulent wind model $V_{hub} = V_{10min, n-yr}$	ESS $H_s = H_{s, n-yr}$	MIS, MUL	ECM $n-yr$ Current	EWLR $n-yr$ Water Level	Loss of electrical network $n = 50$ in general; $n = 100$ for tropical cyclone-prone sites (See Note 1 for further guidance)	S	A
	6.3	EWM Turbulent wind model $V_{hub} = V_{10min, n-yr}$	ESS $H_s = H_{s, n-yr}$	MIS, MUL	ECM $n-yr$ Current	NWLR	Extreme yaw misalignment $n = 1$	S	N
	6.4	NTM $V_{hub} \leq V_{10min, n-yr}$	NSS Joint prob. distribution of H_s, T_p, V_{hub}	MIS, MUL	NCM	NWLR or \geq MSL	$n = 1$	F	FDF

Design Condition	DLC	Wind Condition	Waves	Wind and Wave Directionality	Sea Currents	Water Level	Other Conditions	Type of Analysis	Safety Factor
7) Parked and fault conditions	7.1	EWM Turbulent wind model $V_{hub} = V_{10min}, n - yr$	ESS $H_S = H_{S,n - yr}$	MIS, MUL	ECM n -yr Current	NWLR	$n = 1$	S	A
	7.2	NTM $V_{hub} \leq V_{10min}, n - yr$	NSS Joint prob. distribution of H_S, T_p, V_{hub}	MIS, MUL	No current	NWLR or \geq MSL	$n = 1$	F	FDF
8) Transport, assembly, maintenance and repair	8.1	To be defined by the Fabricator and/or Owner							T
	8.2	EWM Turbulent wind model $V_{hub} = V_{10min}, n - yr$	ESS $H_S = H_{S,n - yr}$	COD, UNI	ECM n -yr Current	NWLR	$n = 1$	S	A
	8.3	NTM $V_{hub} \leq V_{10min}, n - yr$	NSS Joint prob. distribution of H_S, T_p, V_{hub}	MIS, MUL	NCM	NWLR or \geq MSL	No grid during installation period $n = 1$	F	FDF
	8.4	To be defined by the Fabricator or Owner							FDF

Notes:

- 1 The reduction of the return period of 100 years to 50 years in tropical cyclone prone areas may be acceptable provided additional safety factors are applied or robustness assessments for the Survival Load Cases are performed to justify such reduction.
- 2 The descriptions of the design load cases in the table are provided in 4-2/3. The symbols and abbreviations used in the table are summarized as follows.

COD	co-directional (aligned) wind and wave direction	F	fatigue (4-2/3)
DLC	design load case	S	strength (4-2/3)
ECD	extreme coherent gust with direction change (3-2/11.9)	N	normal (4-2/3)
ECM	extreme current model (3-4/5)	A	abnormal (4-2/3)
EDC	extreme direction change (3-2/11.7)	T	transport, assembly, maintenance and repair (4-2/3)
EOG	extreme operating gust (3-2/11.3)	FDF	fatigue design factor (Chapter 5)
ESS	extreme sea state (3-3/7)	H_s	significant wave height
ETM	extreme turbulence model (3-2/11.5)	$H_{s^2, n-yr}$	significant wave height of the extreme sea state with a return period of n years (3-3/7)
EWLR	extreme water level range (3-5/5)	H_{n-yr}	extreme wave height with a return period of n years (3-3/7)
EWM	extreme wind speed model (3-2/11.1)	$H_{s,SSS}$	significant wave height of the severe sea state with a return period of n years (3-3/5)
EWS	extreme wind shear (3-2/11.11)	n	return period of an extreme environmental condition (4-2/3)
MIS	misaligned wind and wave directions	T_p	peak period of wave spectrum
MSL	mean sea level (3-5/1 FIGURE 1)	$V_{1min, n-yr}$	1-minute mean wind speed at hub height with a return period of n years (3-2/11)
MUL	multi-directional wind and wave	$V_{10min, n-yr}$	10-minute mean wind speed at hub height with a return period of n years (3-2/11)
NCM	normal current model (3-4/3)	$V_{3sec, n-yr}$	3-second mean wind speed at hub height with a return period of n years (3-2/11)
NTM	normal turbulence model (3-2/9.3)	V_{hub}	10-minute mean wind speed at hub height
NWLR	normal water level range (3-5/3)	$V(z_{hub})$	steady wind speed at hub height, z_{hub}
NWP	normal wind profile model (3-2/9.1)	V_{in}	cut-in wind speed (1-4/3.7)
NSS	normal sea state (3-3/3)	V_{out}	cut-out wind speed (1-4/3.9)
SSS	severe sea state (3-3/5)	V_r	rated wind speed (1-4/3.39)
UNI	uni-directional wind and wave directions	$V_r \pm 2 \text{ m/s}$	(66 ft/s) sensitivity to the wind speeds in the given range (4-2/3)

TABLE 2
Design Load Cases for Ice Conditions (1 July 2020)

Design Condition	DLC	Ice Condition	Wind Condition	Water Level	Type of Analysis	Safety Factor
Power Production	D1	Horizontal load from temperature fluctuations	NTM $V_{hub} = V_r \pm 2\text{m/s}(6.6\text{ft/s})$ and V_{out} Wind Speed resulting in maximum thrust	NWLR	S	N
	D2	Horizontal load from water fluctuations or arch effect	NTM $V_{hub} = V_r \pm 2\text{m/s}(6.6\text{ft/s})$ and V_{out} Wind Speed resulting in maximum thrust	NWLR	S	N
	D3	Horizontal load from moving ice floe at relevant velocities $h_{ice} = h_{ice, 50\text{-yr}}$ in open sea $h_{ice} = h_{ice, m}$ for land-locked waters	NTM $V_{hub} = V_r \pm 2\text{m/s}(6.6\text{ft/s})$ and V_{out} Wind Speed resulting in maximum thrust	NWLR	S	N
	D4	Horizontal load from moving ice floe at relevant velocities $h_{ice} = h_{ice, 50\text{-yr}}$ in open sea $h_{ice} = h_{ice, m}$ for land-locked waters	NTM $V_{in} \leq V_{hub} \leq V_{out}$	NWLR	F	FDF
	D5	Vertical force from fast ice covers due to water level fluctuations	No wind load applied	NWLR	S	N
Parked	D6	Pressure from hummocked ice and ice ridges	EWM $V_{hub} = V_{10\text{min}, 1\text{-yr}}$	NWLR	S	N
	D7	Horizontal load from moving ice floe at relevant velocities $h_{ice} = h_{ice, 50\text{-yr}}$ in open sea $h_{ice} = h_{ice, m}$ for land-locked waters	NTM $V_{hub} \leq V_{10\text{min}, 1\text{-yr}}$	NWLR	F	FDF

Notes:

The symbols and abbreviations used in the table are described in the Notes of 4-2/3 TABLE 1 in addition to those summarized as follows.

h_{ice} ice thickness

$h_{ice, 50-yr}$ extreme ice thickness with a return period of 50 years

$h_{ice, m}$ long term mean value of the annual maximum ice thickness

5 Definition of Survival Load Cases (SLCs) (1 July 2020)

Where the robustness assessment of the Support Structure in a tropical cyclone region is chosen to demonstrate the ultimate strength capacity of the Support Structure, the Survival Load Cases (SLCs) defined in this Subsection are to be applied.

As a minimum, the SLCs specified in 4-2/Table 3 are to be assessed. The probability of joint occurrence of environmental parameters are to be taken into account when establishing survival metocean conditions, with the consideration of the peak wind, peak wave and peak current condition (see, e.g., API RP 2MET). The effect of environmental loads is to be combined with the effect of dead loads, live loads and deformation loads. Combinations of the load effects that produce the most unfavorable effects on the Support Structure are to be used to assess the design adequacy.

The safety factors applicable for the survival load case are described in Chapter 5. The differentiation of ‘N’, ‘A’ and ‘T’ turbine operating conditions is not applicable to the survival load cases.

TABLE 3
Survival Load Cases – Tropical Cyclone Regions

<i>Design Condition</i>	<i>Wind Condition</i>	<i>Waves</i>	<i>Wind and Wave Directionality</i>	<i>Sea Currents</i>	<i>Water Level</i>	<i>Other Conditions</i>
Parked (standing still or idling)	SurWM $V_{hub} = V_{10min, n-yr}$	SurSS $H_s = H_{s, n-yr}$	MIS, MUL	SurCM n -yr Currents	SurWLR n -yr Water Level	Intact blade(s)
	SurWM $V_{hub} = V_{10min, 500-yr}$	SurSS $H_s = H_{s, 500-yr}$	MIS, MUL	SurCM 500-yr Currents	SurWLR 500-yr Water Level	Damaged blade(s), where relevant ^[1]
	SurWM $V_{hub} = V_{10min, m-yr}$	SurSS $H_s = H_{s, m-yr}$	MIS, MUL	SurCM m -yr Currents	SurWLR m -yr Water Level	Loss of electrical network ^[2]

Notes:

- 1 “Damaged blade(s)” case is to be assessed if one turbine blade or multiple turbine blades cannot remain intact under the storm wind condition with a return period of 500 years (i.e., $n < 500$ years)
- 2 For the ‘Loss of electrical network’ case, the return period ‘ m ’ for extreme environmental conditions is selected such that the joint event of the loss of yaw power and controls during the extreme environmental conditions has the probability of 1/500, or equivalently a return period of 500 years. The value of ‘ m ’ should be selected and justified by the designer. For sites expected to experience a loss of network power, or in the absence of information defining the network reliability, a return period of 500 years for environmental conditions may be used as a conservative value, unless back-up power is to be assessed.
- 3 The symbols and abbreviations used in the table are summarized as follows:
 n -yr : maximum return period (n years) of the storm wind condition that turbine blades can sustain and remain intact or 500 years, whichever is less
 m -yr : maximum return period (m years) of the storm wind condition as defined above in item (2)
 $H_{s,n\text{-yr}}$: significant wave height with a return period of n years
 $H_{s,500\text{-yr}}$: significant wave height with a return period of 500 years
 $V_{10\text{min},n\text{-yr}}$: 10 minute mean wind speed at hub height with a return period of n years
 $V_{10\text{min},500\text{-yr}}$: 10 minute mean wind speed at hub height with a return period of 500 years
SurWM : survival wind model (3-2/13.1)
SurSS : survival sea state (3-3/9)
SurCM : survival current model (3-4/7)
SurWLR : survival water level range (3-5/7)
Other symbols and abbreviations used in the table are defined in the Notes of 4-2/Table 1.

CHAPTER 4 Loads

SECTION 3 Determination of Environmental Loads

1 General (15 January 2013)

Environmental loads are to be determined using analytical methods compatible with the environmental condition models established in compliance with Chapter 3. Any recognized load calculation method may be employed provided it has been proven sufficiently accurate in practice, and it is shown to be appropriate to the structure's characteristics and site conditions.

Model or on-site test data can also be used to establish environmental loads.

3 Wind Loads (15 January 2013)

Wind loads and local wind pressures are to be determined on the basis of analytical methods or wind tunnel tests using a representative model of the offshore wind turbine. Static and dynamic wind load effects generated directly by inflowing wind and indirectly by wind generated responses of the RNA are to be taken into account.

3.1 Aerodynamic Loads Generated by the Rotor

Aerodynamic loads induced by airflow passing through the rotor are determined by the mean wind speed and turbulence across the rotor plane, rotor rotational speed, air density, and aerodynamic shapes of wind turbine components, as well as interactive effects such as aero-elasticity and rotational sampling. Aerodynamic loads due to these effects are to be calculated using recognized methods and computer programs.

The wind models in Section 3-2 and the DLCs in Section 4-2 specify the design load conditions for which aerodynamic loads generated by the rotor are to be calculated.

For offshore wind turbines installed in a wind farm, the potential shadow effect and wake effect on the loads are to be considered for both the strength and fatigue analyses. For large wind farms, an increase in the turbulence intensity or terrain roughness is to be taken into account. The mutual influence of offshore wind turbines through the wake interaction behind the rotor is to be considered up to a distance of 10 times of rotor diameter. Reference is made to IEC 61400-1 for the guidance on the wake effect from neighboring offshore wind turbines.

3.3 Wind Forces on Exposed Structural Components

For wind pressure normal to flat surfaces, such as nacelle and boat landing, or normal to the axis of members not having flat surfaces, such as tower and exposed above-water tubular members of the Support Structure, the wind loading can be considered as a constant (steady) force determined as follows:

$$F_w = (\rho/2g)C_sAV^2$$

where

F_w = wind drag load, in N (lb)

ρ = weight density of air, in N/m³ (lb/ft³)

g = acceleration of gravitation, in m/s² (ft/s²)

C_s = shape coefficient

A = projected area of member on a plane normal to the direction of the considered force, in m² (ft²)

V = wind speed at a given elevation above the SWL, in m/s (ft/s)

In the absence of experimental data, values in 4-3/3.3 TABLE 1 for the shape coefficient (C_s) are to be applied.

For any direction of wind approaching to the structure, the wind force on flat surfaces is to be assumed to act normal to the surface. The wind force on cylindrical objects is to be assumed to act in the direction of the wind.

The conversion of mean wind speeds with different averaging time durations and the applicable wind profile are to follow 3-2/3.1.

TABLE 1
Values of C_s (15 January 2013)

Shape	Values of C_s
Sphere	0.40
Cylindrical shape	0.50
Major flat surfaces and overall projected area of the Support Structure	1.00
Isolated structural shapes (cranes, angles, beams, channels, etc.)	1.50
Under deck areas (smooth)	1.00
Under-deck areas (exposed beams and girders)	1.30
Derricks or truss cranes (each face)	1.25
Sides of buildings	1.50

The area of open truss works commonly used for derricks and crane booms may be approximated by taking 30% of the projected area of both the windward and leeward sides with the shape coefficient taken in accordance with 4-3/3.3 TABLE 1.

Where one structural member shields another from direct exposure to the wind, shielding may be taken into account. Generally, two structural components are to be separated by not more than seven times the width of the windward component in order for a reduction to be taken in the wind load on the leeward member.

Where applicable, cyclic loads due to vortex induced vibration (VIV) of structural members exposed to wind loads are to be investigated. Both drag and lift components of load due to vortex induced vibration are to be taken into account. The effects of wind loading on structural members or components that are not normally exposed to wind loads after installation are to be considered, especially during fabrication or transportation phases.

5 Wave Loads (1 July 2020)

A sufficient range of realistic wave periods and wave crest positions relative to the structure are to be investigated to determinate the maximum wave loads on the Support Structure. Consideration is also to be given to other wave induced effects such as wave impact loads, dynamic amplification and fatigue of structural members. The need for analysis of these effects is to be assessed based on the configuration and behavioral characteristics of the Support Structure, the wave conditions and past experience.

For structures composed of members having diameters which are less than 20% of the wave lengths being considered, semi-empirical formulations such as Morison's equation are considered to be an acceptable basis for determining wave loads. For structures composed of members whose diameters are greater than 20% of the wavelengths being considered, or for structural configurations that substantially alter the incident flow field, diffraction forces and the hydrodynamic interaction of structural members are to be accounted for in the design.

The hydrodynamic force acting on a cylindrical member, as given by Morison's equation, is expressed as the sum of the force vectors indicated in the following equation:

$$F = F_D + F_I$$

where

F = hydrodynamic force vector per unit length along the member, acting normal to the axis of the member, in N/m (lb/ft)

F_D = drag force vector per unit length, in N/m (lb/ft)

F_I = inertia force vector per unit length, in N/m (lb/ft)

The drag force vector for a stationary, rigid member is given by:

$$F_D = (\rho/2g)DC_D u_n |u_n|$$

where

ρ = weight density of water, in N/m³ (lb/ft³)

g = acceleration of gravitation, in m/s² (ft/s²)

D = projected width of the member in the direction of the cross-flow component of velocity (in the case of a circular cylinder, D denotes the diameter), in m (ft)

C_D = drag coefficient

u_n = component of the fluid velocity vector normal to the axis of the member, in m/s (ft/s)

$|u_n|$ = absolute value of u_n , in m/s (ft/s)

The inertia force vector for a stationary, rigid member is given by:

$$F_I = (\rho/g)(\pi D^2/4)C_M a_n$$

where

C_M = inertia coefficient based on the displaced mass of fluid per unit length

a_n = component of the fluid acceleration vector normal to the axis of the member, in m/s² (ft/s²)

For the compliant turbine Support Structures which exhibit substantial rigid body oscillations due to the wave action, the modified form of Morison's equation given below may be used to determine the hydrodynamic force.

$$F_D = (\rho/2g)DC_D(un - \bar{u}_n)|u_n - \bar{u}_n|$$

$$F_I = (\rho/g)(\pi D^2/4)a_n + (\rho/g)(\pi D^2/4)C_m(a_n - \bar{a}_n)$$

where

\bar{u}_n = component of the velocity vector of the structural member normal to its axis, in m/s (ft/s)

\bar{a}_n = component of the acceleration vector of the structural member normal to its axis, in m/s² (ft/s²)

C_m = added mass coefficient

$$= C_M - 1$$

For shapes of structural member other than circular cylinders, the term $\pi D^2/4$ in the above equations is to be replaced by the actual cross-sectional area of the shape.

Values of u_n and a_n in Morison's equation are to be determined using a recognized wave theory appropriate to the wave heights, wave periods, and water depth at the installation site. Values for the coefficients of drag and inertia in Morison's equation are to be determined based on model tests, full-scale measurements, or previous studies which are appropriate to the structural configuration, surface roughness, and pertinent flow parameters (e.g., Reynolds number). Reference is made to API RP 2A-WSD for the recommended values.

For structural configurations which substantially alter the incident wave field, diffraction theories of wave loading are to be employed to account for both the incident wave force (i.e., Froude-Kylov force) and the force resulting from the diffraction of the incident wave due to the presence of the structure. The hydrodynamic interaction of structural members is to be taken into account.

For installation sites where the ratio of water depth to **wavelength** is less than 0.25, nonlinear effects of wave action are to be taken into account. This may be fulfilled by modifying linear diffraction theory to account for nonlinear effects or by performing model tests. Wave force calculations are to take into account shallow water effects which may increase the current due to blockage effects and alter wave kinematics.

Where the effect of breaking waves is to be taken into account in the design, guidance for the breaking wave hydrodynamics and the loads exerted by a breaking wave on a structure may be referred to IEC 61400-3-1 (2019), Annex **B and C**.

7 Current Loads (15 January 2013)

Current induced loads on immersed structural members are to be determined based on analytical methods, model test data or full-scale measurements. When currents and waves are superimposed, the current velocity is to be added vectorially to the wave induced particle velocity prior to computation of the total force. Current profiles used in the design are to be representative of the expected conditions at the installation site. Where appropriate, flutter and dynamic amplification due to vortex shedding are to be taken into account.

For calculation of current loads in the absence of waves, the lift force normal to flow direction, and the drag force may be determined as follows.

$$F_L = (\rho/2g)AC_LU^2$$

$$F_D = (\rho/2g)AC_DU^2$$

where

- F_L = total lift force per unit length, in N/m (lb/ft)
- F_D = total drag force per unit length, in N/m (lb/ft)
- ρ = weight density of water, in N/m³ (lb/ft³)
- g = acceleration of gravitation, in m/s² (ft/s²)
- A = projected area per unit length in a plane normal to the direction of the force, in m²/m (ft²/ft)
- C_L = lift coefficient
- C_D = drag coefficient (see 4-3/5)
- U = local current velocity, in m/s (ft/s)

For long cylindrical members with large length-diameter ratios, lift force may become significant and is to be considered in the design. The source of C_L values employed is to be documented.

Vortex induced vibration (VIV) is a resonant response caused by vortex shedding at resonant frequencies in current. Effects of VIV on fatigue and increased drag loads are to be suitably assessed for structural members. Dynamic amplification due to vortex shedding is to be considered if deemed necessary.

9 Ice and Snow Accumulation Induced Loads

At locations where an offshore wind turbine is subjected to ice and snow accumulation, increased weight and change in effective area of structural members due to accumulated ice and snow are to be considered. Particular attention is to be paid to possible increases in aerodynamic and hydrodynamic loading due to the change in size and surface roughness of both non-rotating and rotating parts of an offshore wind turbine caused by ice and snow accumulation.

11 Earthquake Loads (1 July 2020)

For the Bottom-founded Offshore Wind Turbine located in seismically active areas, the Strength Level and Ductility Level earthquake induced ground motions (see 3-6/9) are to be determined based on seismic data applicable to the installation site.

Earthquake ground motions are to be described by either applicable ground motion records or response spectra consistent with the return period appropriate to the design life of the Bottom-founded Offshore Wind Turbine. Available standardized spectra applicable to the region of the installation site are acceptable provided such spectra reflect site-specific conditions affecting frequency content, energy distribution, and duration. These conditions include:

- The type of active faults in the region
- The proximity of the site to the potential source faults
- The attenuation or amplification of ground motion between the faults and the site
- The soil conditions at the site

The ground motion description used in the design is to consist of three components corresponding to two orthogonal horizontal directions and the vertical direction. All three components are to be applied to the structure simultaneously.

When a standardized response spectrum, such as the one given in the API RP 2A-WSD, is used for structural analysis, input values of ground motion (spectral acceleration representation) to be used are not to be less severe than the following.

- 100% in both orthogonal horizontal directions
- 50% in the vertical direction

When three-dimensional, site-specific ground motion spectra are developed, the actual directional accelerations are to be used. If single site-specific spectra are developed, accelerations for the remaining two orthogonal directions are to be applied in accordance with the factors given above.

If time history method is used for structural analysis, at least three sets of ground motion time histories are to be employed. The time histories are used to account for the potential sensitivity of the structure's response to variations in the phasing of the ground motion records.

Structural appurtenances and turbine equipment are to be designed to resist earthquake induced accelerations at their foundations.

As appropriate, effects of soil liquefaction, shear failure of soft mud and loads due to acceleration of the hydrodynamic added mass by the earthquake, submarine slide, tsunamis and earthquake generated acoustic shock waves are to be taken into account.

13 Marine Growth

The following effects of anticipated marine growth are to be accounted for in the design.

- Increase in hydrodynamic diameter
- Increase in surface roughness used in the determination of hydrodynamic coefficients (e.g., lift, drag and inertia coefficients)
- Increase in dead load and inertial mass

The amount of accumulation assumed for a specific design is to reflect the extent of and interval between cleaning of submerged structural parts.

15 Ice Loads (1 July 2020)

Ice loads acting on the Support Structure may include both static and dynamic loads. Static loads can be generated by temperature fluctuations, ice features lodged against the structure, or changes in water level in a fast ice cover. Dynamic loads are normally caused by moving ice interactions with the Support Structure.

The methods to determine ice loading on the Support Structure of an offshore wind turbine are described in Annex D of IEC 61400-3-1 (2019). Further reference is made to API RP 2N and ISO 19906.

The global forces exerted by ice on the Support Structure as whole and local concentrated loads on structural elements are to be considered. The effects of rubble piles on the development of larger areas and their forces on the Support Structure need to be considered. Possible ice jamming between legs is to be accounted for where the Support Structure is designed to consist of multiple legs.

Where relevant, liquefaction of the underlying soil due to repetitive compressive failures of the ice against the Support Structure is to be taken into account.

CHAPTER 5 Structure and Foundation Design

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CHAPTER 5 Structure and Foundation Design

SECTION 1 General Design Requirements

1 Overview (1 July 2020)

This Section provides general requirements to be applied to the structural and foundation design of the Bottom-founded Offshore Wind Turbine. The criteria in Sections 2 through 5 of Chapter 5 dealing specifically with the design of steel structures, concrete structures, and foundation are to be applied along with the requirements of this Section.

Wherever references in Chapter 5 are made to the API RP 2A, other applicable recognized industry standards such as ISO standards may also be used.

Design documentation of structures and foundations to be submitted for review is specified in 1-1/9.

The design assessment of design life extension and reuse of the existing Support Structure of an offshore wind turbine is to be in accordance with Section 4-1-2 of the ABS *Rules for Building and Classing Offshore Installations*, with the exception that the load, strength, and fatigue calculations are to follow Chapters 4 and 5 of this Guide.

The design criteria in Chapter 5 are to be applied in the design of the substructure and the foundation of the Bottom-founded Offshore Wind Turbine. The effect of the RNA and tower on the substructure and the foundation design is to be considered.

Compliance of the tower with the design criteria in this Chapter for the substructure is required for the following situations (see 1-1/3):

- The optional **RNA** notation is requested by the Owner
- The optional **RNA** notation is not requested by the Owner, however the tower and its connection to the RNA and/or to the substructure are not included in the wind turbine type certificate

3 Format of Design Approach (15 January 2013)

The design requirements of this Guide are generally specified in terms of the Working Stress Design (WSD) approach for steel components of the Support Structures and the Load and Resistance Factor Design (LRFD) approach for concrete components of the Support Structures. For steel components of the Support Structures, the use of alternative design criteria based on the LRFD approach is to be in accordance with 5-2/9.5. In addition, consideration is to be given to the serviceability of the structure relative to excessive deflection, vibration and, in the case of concrete, cracking.

5 General Design Considerations (15 January 2013)

5.1 Design Load Conditions

The steel Support Structure of an offshore wind turbine is to be designed and analyzed for the loads to which it is likely to be exposed during construction, transportation, installation and in-service operations. Loadings to be investigated for in-service operations are to include at least those relating to realistic operating and environmental conditions combined with dead loads, live loads and deformation loads that are appropriate to the function and operation of an offshore wind turbine.

Load combinations are to reflect the design load conditions as specified in Chapter 4. With reference to Chapters 3 and 4, both stochastic and deterministic formats are used to establish the design load conditions. Where relevant, consideration is to be given to the effect of dynamic amplification. The influence of less severe environmental loads in combination with operational loads is to be investigated for their potential to produce maximum peak stresses in the Support Structure.

For the Support Structures located in seismically active areas, earthquake loads (see 3-6/9 and 4-3/11) are generally to be combined with dead loads, live loads and deformation loads appropriate to the offshore wind turbine operation and function which may be occurring at the onset of an earthquake.

5.3 Design Life (1 July 2020)

The design life of the Bottom-founded Offshore Wind Turbine is not to be less than 20 years.

Upon the request by the Owner, a shorter design life may be accepted by ABS for the purpose of demonstrating viability of a new design concept or conducting pilot operations.

Continuance of classification beyond the design life is subject to the survey requirements in 7-2/1.21 and additional engineering analyses (see 1-1/5.3.2).

5.5 Air Gap

A minimum air gap of 1.5 m (5 ft) is to be provided between the 50-year return maximum wave crest elevation above the highest still water level (HSWL, see 3-5/1 FIGURE 1) and the lowest edge of the Support Structure for which wave forces are not included in the design. Consideration is to be given to the effect of initial and expected long-term settlements of the Support Structure, wave run-up, tilting of the Support Structure and, where appropriate, tsunamis. Local wave crest elevation is to be taken into account as appropriate.

Appurtenances and localized components of the Support Structure are to be designed as appropriate to the local increases in wave pressure due to irregularity of waves and proximity to the Support Structure. The local design wave pressures on appurtenances are not to be less than those used for global structure design at the same elevation.

5.7 Structural Dynamic Properties (1 July 2020)

The ratio of the natural frequencies of the Support Structure, including the tower, substructure and foundation, to the excitation frequencies of the various sources is to be determined. Excitation frequencies are to include rotor rotation frequencies, blade passing frequencies, natural frequencies of blades and other RNA components, and design wave periods. The overall damping contributed from various sources such as aerodynamic damping, hydrodynamic damping, structural damping and soil and foundation damping is to be determined and appropriately taken into account **when** deriving structural dynamic properties. Consideration is also to be given to the oscillatory loading on the Support Structure generated by vortex-induced vibrations transverse to the wind or current direction. Variations of soil properties and the possible occurrence of scour, corrosion, marine growth and sand movement during the design life may change the natural frequencies of the Support Structure and are to be suitably taken into account in the design.

5.9 Long-Term and Secondary Effects

Consideration is to be given to the following effects, as appropriate to the planned Support Structure of an offshore wind turbine:

- i) Local vibration due to machinery, equipment and vortex shedding
- ii) Stress concentrations at critical joints
- iii) Secondary stresses induced by large deflection
- iv) Cumulative fatigue
- v) Corrosion
- vi) Abrasion due to ice
- vii) Freeze-thaw action on concrete and coatings

5.11 Zones of Exposure (1 July 2020)

Measures taken to mitigate the effects of corrosion are to be specified and described by the following definitions for corrosion protection zones.

- i) *Submerged Zone.* That part of the Support Structure below the Splash Zone.
- ii) *Splash Zone.* That part of the Support Structure as defined in 1-2/3.45. Characteristically, the Splash Zone is not easily accessible for field painting, nor protected by cathodic protection.
- iii) *Atmospheric Zone.* That part of the Support Structure above the Splash Zone.

Additionally, for offshore wind turbines located in areas subject to floating or submerged ice, the portion of the Support Structure expected to come into contact with floating or submerged ice is to be designed with consideration for such contact.

7 Considerations for Particular Types of Substructure (1 July 2020)

7.1 General (1 July 2020)

Specific design considerations listed in this Subsection are to be taken into account for particular types of **substructure** of an offshore wind turbine as additional factors that affect the safety and performance of the structure. These design considerations are not intended to supplant or modify other criteria specified in this Guides.

7.3 Pile-Supported Steel Structures (1 July 2020)

Consideration is to be given to the following effects, as appropriate to the **substructure**:

- i) The soil-pile interaction and the loads imposed on the **substructure** during towing and launching are to be considered.
- ii) Carefully controlled installation procedures are to be developed so that the bearing loads of the structure on the soil are kept within acceptable limits until the piles are driven.
- iii) As applicable, special procedures are to be used to handle long, heavy piles until they are self-supporting in the soil. Pile driving delays are to be minimized to avoid setup of the pile sections.
- iv) Natural periods of the **integrated RNA, tower, substructure, and foundation** are to be checked to verify that the **Bottom-founded Offshore Wind Turbine** is not in resonance with excitation loads having significant energy contents.
- v) Instability of structural members due to submersion is to be considered, with due account for second-order effects produced by factors such as geometrical imperfections.

- vi) Connections other than welded joints such as clamps, connectors and bolts, joining diagonal braces to the column or piles to the substructure, the strength and fatigue resistance are to be assessed by analytical methods or testing.

7.5 Concrete or Steel Gravity Structures (1 July 2020)

Considerations are to be given to the following effects, as appropriate to the **substructure**:

- i) The procedure for transporting and positioning the structure and the accuracy of measuring devices used during these procedures are to be documented.
- ii) Effects of repeated loadings on soil properties, such as pore pressure, water content, shear strength and stress strain behavior, are to be investigated.
- iii) Soil reactions against the base of the structure during installation are to be evaluated. Consideration is to be given to the occurrence of point loading caused by sea floor irregularities. Suitable grouting between base slab and sea floor may be employed to reduce concentration of loads.
- iv) The strength and durability of construction materials are to be maintained. Where sulphate attack is anticipated, mitigation measures are to be implemented by choosing appropriate cements, incorporating pozzolans in the mix, or applying suitable coatings on the surfaces.
- v) Instability of structural members due to submersion is to be considered, with due account for second-order effects produced by factors such as geometrical imperfections.
- vi) Where necessary, protection against horizontal sliding along the sea floor is to be provided by means of skirts, shear keys or equivalent means.
- vii) The long-term resistance to abrasion, cavitation, freeze-thaw durability and strength retention of the concrete are to be considered. Means are to be provided to minimize reinforcing steel corrosion.
- viii) Provision is to be made to maintain adequate negative buoyancy at all times to resist the uplift forces from wind, waves, currents, and overturning moments.

7.7 Concrete-Steel Hybrid Foundation Structures

Where necessary, the underside of the concrete base is to be provided with skirts or shear keys to resist horizontal loading. Steel or concrete keys or equivalent means may be used in the design.

The steel portions of a steel-concrete hybrid structure are to be designed in accordance with Chapter 5, Section 2; the concrete portions are to be designed in accordance with Chapter 5, Section 3. Any effects of the hybrid structure interacting on itself in areas such as corrosion protection are to be considered. Special attention is to be paid to the design of the connections between steel and concrete components.

Applicable design considerations in 5-1/7.5 for concrete bases are also to be taken into account.

7.9 Self-Elevating Unit to be Used as the **Substructure** (1 July 2020)

A self-elevating mobile offshore unit converted to the site dependent **substructure** of an offshore wind turbines is to be designed in accordance with this Guide in conjunction with the *ABS Rule for Building and Classing Mobile Offshore Units*, wherever applicable.

When selecting a unit for a particular site, due consideration is to be given to soil conditions at the installation site. The bearing capacity and sliding resistance of the foundation are to be investigated. The foundation design is to be in accordance with 5-4/7. As applicable, the footprints left by a self-elevating unit and scour are to be considered in the foundation design.

In the structural analysis, the leg to hull connections and soil/structure interaction are to be properly considered. The upper and lower guide flexibility, stiffness of the elevating/holding system, and any special details regarding its interaction with the leg are to be taken into consideration. For units with spud

cans, the legs may be assumed pinned at the reaction point. For mat supported units, the soil structure interaction may be modeled using discrete elastic elements (springs).

While used as the site dependent **substructure**, the calculated loads are to demonstrate that the maximum holding capacity of the jacking system will not be exceeded.

Units with spudcans are to be pre-loaded during installation in order to minimize the possibility of significant settlement under severe storm conditions.

CHAPTER 5 Structure and Foundation Design

SECTION 2 Steel Structures

1 General (1 July 2020)

The requirements of this Section are to be applied in the design and analysis of the steel **substructure** of an offshore wind turbine. Section 5-1 is to be consulted regarding general design requirements. Items to be considered in the design of welded connections are specified in Section 2-2.

1.1 Materials (1 July 2020)

The requirements of this Section are specified for the **substructure** constructed of steel, which is manufactured and has properties as specified in Chapter 2, Section 1. Where it is intended to use steel or other materials having properties differing from those specified in Chapter 2, Section 1, their applicability will be considered by ABS upon reviewing the specifications of alternative materials and the proposed methods of fabrication.

1.3 Corrosion Protection (1 July 2020)

Materials are to be protected from corrosion by using a corrosion protection system including coatings. The system is to be effective from the time the **substructure** is initially placed on site. Where the sea environment contains unusual contaminants, any special corrosive effects of such contaminants are to be considered. For the design of protection systems, reference is to be made to the publications from NACE International: SP0176 and SP0108, or other recognized standards.

1.5 Access for Inspection (1 July 2020)

In the design of the **substructure**, consideration is to be given to providing access for inspection during construction and, to the extent practicable, for survey after construction. Any openings on the **substructure** for the purpose of providing access to an offshore wind turbine are to be evaluated to verify there is no adverse effect on the integrity of the structure.

3 Design Criteria (1 July 2020)

The steel **substructure** of an offshore wind turbine is to be designed and analyzed for the loads as specified in 5-2/5. The resultant structural responses of the **substructure** are to satisfy the acceptance criteria in 5-2/9 through 5-2/23.

5 Loading Conditions (1 July 2020)

Loading conditions for the structural analysis and design of the steel **substructure** are to be determined in accordance with 5-1/5.1. The Design Load Cases (DLCs) in 4-2/3 TABLE 1 are to be assessed as a

minimum requirement of design load conditions. DLCs for ice conditions are to be considered in accordance with 4-2/3 for the installation site where ice is expected to occur. Where the robustness assessment of the substructure and the foundation in a tropical cyclone region is performed, the Survival Load Cases (SLCs) in 4-2/Table 1 are to be assessed as a minimum.

7 Structural Analysis (1 July 2020)

The following is general guidance on choosing appropriate approaches to perform structural analyses for the substructure of an offshore wind turbine. The designer is to verify that the structural analysis method is suitable for specific structural behaviors and can lead to accurate analysis results.

- i) The nature of loads and loading combinations as well as the local environmental conditions are to be taken into consideration in the selection of design methods. Methods of analysis and their associated assumptions are to be compatible with the overall design principles. When assessing structural instability as a possible mode of failure, the effects of initial stress and geometric imperfections are to be taken into account. Construction tolerances are to be consistent with those used in the structural stability assessment.
- ii) Dynamic analyses are to be considered to assess the effects of environmental or other types of loads where dynamic amplification is expected. When performing fatigue analyses, a long-term distribution of the stress range, with proper consideration of dynamic effects, is to be obtained for relevant loadings anticipated during the design life of the substructure.
- iii) The interaction among environmental loadings and between external loads and structural responses are to be assessed by applying environmental loads simultaneously to the substructure. Other rational methods for calculating loadings are acceptable provided it is demonstrable that the adopted method leads to an equivalent design.
- iv) Soil-pile interaction can affect overall dynamic behavior of the substructure. The simulation of foundation soil-pile interaction in the structural analysis is to be properly carried out based on site-specific soil conditions.
- v) For static loads, plastic methods of design and analysis can be employed only when the properties of the steel and the connections are such that they exclude the possibility of brittle fracture, allow for formation of plastic hinges with sufficient plastic rotational capability, and provide adequate fatigue resistance.
- vi) Under dynamic loads, when plastic strains may occur, the considerations specified in v) are to be satisfied and any buckling and destabilizing nonlinear effects are to be taken into account.
- vii) When considering the effects of earthquakes in seismically active areas (see 3-6/9 and 4-3/11), dynamic analyses are to be performed using the recognized methods, such as those recommended in API RP 2A.
- viii) Where the Survival Load Cases in accordance with 4-2/5 are assessed, the ultimate strength analysis method provided in API RP 2SIM is to be applied.

9 Strength Design Criteria

9.1 General (1 July 2020)

The design of the steel substructure of an offshore wind turbine can be based on either the Working Stress Design (WSD) approach or the Load and Resistance Factor Design (LRFD) approach as specified in 5-2/9.3 and 5-2/9.5, respectively. However, it is not permitted to mix elements of these two approaches in the design for the same structural component in the substructure.

The strength design criteria specified in this Subsection are to be applied to those design load cases relevant to the strength assessment. A minimum set of Design Load Cases (DLCs) for the strength

assessment is defined in 4-2/3 TABLE 1 and, if applicable, 4-2/3 TABLE 2 and 4-2/Table 3, where the ‘Type of Analysis’ column is denoted ‘S’.

9.3 Working Stress Design (WSD) Approach (1 July 2020)

When the strength design of the steel substructure is based on the WSD approach, the design acceptance criteria are to be expressed in terms of appropriate basic allowable stresses in accordance with the requirements specified in this Subsection. Linear, elastic methods can be employed in the analysis of the substructure provided proper measures are taken to prevent general and local buckling failure, and the interaction between soil and structure is adequately considered.

The loading conditions for the strength analysis are to be in accordance with 5-2/5. A factor of 1.0 applies to all categories of load. When a live load is considered a favorable load that relieves total load responses, the minimum value of this live load, as appropriate to the operation mode being considered, is to be used in the load combination.

For the Design Load Cases, the factors of safety specified in this Subsection are to be applied in conjunction with the normal (N) and abnormal (A) design conditions as well as the design conditions (T) related to transport, installation, maintenance and repair operations, as defined in 4-2/3.

Where the Survival Load Cases in accordance with 4-2/5 are assessed, structural members may reach their ultimate capacities or beyond, provided the global strength of the substructure is not compromised.

9.3.1 Individual Stresses in Structural Members

Individual stress components or direct combinations of such stresses in a structural member are not to exceed the allowable stress as obtained from the following equation:

$$F_{allowable} = F_y / F.S.$$

where

$F_{allowable}$ = allowable stress

F_y = specified minimum yield strength, as defined in the *ABS Rules for Materials and Welding (Part 2)*

$F.S.$ = factor of safety

- For the normal design conditions (designated ‘N’ in the column entitled as ‘Safety Factor’ in 4-2/Table 1 and 4-2/Table 2)

= 1.5 for axial or bending stress

= 2.5 for shear stress

- For the abnormal design conditions (designated ‘A’ in the column entitled as ‘Safety Factor’ in 4-2/Table 1 and 4-2/Table 2)

= 1.25 for axial or bending stress

= 2.0 for shear stress

- For the design load conditions involving the transport, assembly on site, maintenance, and repair operations (DLC 8.1 in 4-2/3 TABLE 1)

= 1.67 for axial or bending stress

= 2.75 for shear stress

9.3.2 Buckling Strength of Structural Members Subject to a Single Action

Buckling is to be considered for a structural element subject to compressive axial load or bending moment. The computed compressive or bending stress is not to exceed the allowable stress as obtained from the following equation:

$$F_{allowable} = F_{cr} / F.S.$$

where

$F_{allowable}$ = allowable stress

F_{cr} = critical buckling strength of a structural member under axial compression or critical bending strength of a structural member under bending moment, as defined in Section 2 of the *ABS Guide for Buckling and Ultimate Strength Assessment for Offshore Structures*

$F.S.$ = factor of safety

- For the normal design conditions (designated 'N' in the column entitled 'Safety Factor' in 4-2/ Table 1 and 4-2/ Table 2)

$$= 1.5 / \psi$$

- For the abnormal design conditions (designated 'A' in the column entitled 'Safety Factor' in 4-2/ Table 1 and 4-2/ Table 2)

$$= 1.25 / \psi$$

- For the design load conditions involving the transport, assembly on site, maintenance, and repair operations (DLC 8.1 in 4-2/ Table 1)

$$= 1.67 / \psi$$

ψ = adjustment factor, as defined in Subsection 1/11 of the *ABS Guide for Buckling and Ultimate Strength Assessment for Offshore Structures*

9.3.3 Structural Members Subject to Combined Axial Load and Bending

Structural members under axial tension or compression in combination with bending are to be designed according to Section 2 of the *ABS Guide for Buckling and Ultimate Strength Assessment for Offshore Structures* in conjunction with the utilization factors, which are the reciprocals of the corresponding safety factors specified in 5-2/9.3.2.

9.3.4 Allowable Stress of Plated Structures

For plated structures where the equivalent stress is determined using the von Mises equivalent stress criterion, the equivalent stress is not to exceed the allowable stress as obtained from the following equation:

$$F_{allowable} = F_y / F.S.$$

where

$F_{allowable}$ = allowable stress

F_y = specified minimum yield strength, as defined in the *ABS Rules for Materials and Welding (Part 2)*

$F.S.$ = factor of safety

- For the normal design conditions (designated 'N' in the column entitled 'Safety Factor' in 4-2/ Table 1 and 4-2/ Table 2)

$$= 1.33$$

- For the abnormal design conditions (designated 'A' in the column entitled 'Safety Factor' in 4-2/Table 1 and 4-2/Table 2)

$$= 1.11$$

- For the design load conditions involving the transport, assembly on site, maintenance and repair operations (DLC 8.1 in 4-2/3 TABLE 1)

$$= 1.5$$

9.3.5 Buckling Strength of Plated Structures

The buckling strength of plated structures is to be designed according to the *ABS Guide for Buckling and Ultimate Strength Assessment for Offshore Structures* in conjunction with the utilization factors, which are the reciprocals of the corresponding safety factors specified in 5-2/9.3.2.

9.5 Load and Resistance Factor Design (LRFD) Approach (1 July 2020)

In lieu of the WSD approach described in 5-2/9.3, the design of the steel substructure can also be based on the LRFD approach.

The load combination is to be in accordance with 5-2/5 in general.

For the Design Load Cases specified in 5-2/3, the partial safety factors (γ_f) specified in 5-2/9.5 TABLE 1 are to be applied to environmental loads.

A partial safety factor of 1.0 is to be applied to dead loads, live loads, and deformation loads, where they are combined with the design environmental loads. Where a dead load, a live load, or a deformation load is considered as a favorable load that relieves total load responses, a partial safety factor of 0.9 is to be applied. Where a live load is considered a favorable load, the minimum value of this live load is to be used in the load combination.

TABLE 1
Partial Safety Factors (γ_f) for Environmental Loads (1 July 2020)

Normal (N)	Abnormal (A)	Transport, Assembly, Maintenance and Repair (T)
1.35	1.1	1.5

Note: The design conditions represented by N, A and T are defined in 4-2/3.

Alternatively, the partial load factors specified in IEC 61400-3-1 can be applied to the load effects determined by the dynamic analysis of the Bottom-founded Offshore Wind Turbine using unfactored loads.

For design of tubular members and their connections (joints), resistance factors, member strength capacity equations, and strength criteria are to be determined in accordance with API RP 2A-LRFD or ISO 19902.

For structural members with other types of sections within the scope of the AISC *Steel Construction Manual* (LRFD part), resistance factors and member strength capacity equations are to be determined based on the relevant design requirements specified in that standard.

For the ultimate capacity of flat plate and the cylindrical shell structure, reference is made to the *ABS Guide for Buckling and Ultimate Strength Assessment for Offshore Structures* or other recognized industry standards. A minimum resistance factor of $1.05/\psi$, where the adjustment factor ψ is defined in 5-2/9.3.2, is to be applied.

Where the resistance concerns bolted connections and fillet and partial penetration welds, the minimum resistance factor of 1.30 is to be applied.

Where the Survival Load Cases in accordance with 4-2/5 are assessed, all partial safety factors are to be taken as 1.0, and structural members can reach their ultimate capacities or beyond, provided the global strength of the substructure is not compromised.

11 Structural Response to Earthquake Loads (1 July 2020)

Structures located in seismically active areas are to be designed to possess adequate strength and stiffness to withstand the effects of the Strength Level earthquake, as well as sufficient ductility to remain stable during rare motions of greater severity associated with the Ductility Level earthquake. Refer to 3-6/9 and 4-3/11 for the definitions of the Strength Level and Ductility Level earthquakes as well as earthquake loads. The sufficiency of the structural strength and ductility is to be demonstrated by strength and, as required, ductility analyses.

For the Strength Level earthquake, the strength analysis is to demonstrate that the structure is adequately sized for strength and stiffness to maintain all nominal stresses within their yield or buckling limits.

In the ductility analysis, it is to be demonstrated that the structure has the capability of absorbing the energy associated with the Ductility Level earthquake without reaching a state of incremental collapse.

The safety factors for the strength design of structural members, as those defined in 5-2/9.3 for the normal design conditions, are to be reduced by a factor of 1.5, except that for the plated structures designed to 5-2/9.3.4, the allowable stress is to be taken as the specified minimum yield strength (F_y). In the case that the design is based on the LRFD approach, the load factors for all load types considered in the load combinations are to be taken as 1.0.

Pile-soil performance and pile design requirements are to be determined based on case-by-case studies.

13 Fatigue Assessment (1 July 2020)

The fatigue assessment is to be performed to verify adequate capacity against fatigue failure within the design life of the Bottom-founded Offshore Wind Turbine.

The fatigue assessment is to be performed for structural members and joints where fatigue is a probable mode of failure, or for which experience is insufficient to justify safety from possible cumulative fatigue damage. Emphasis is to be given to structural members and connections that are difficult to inspect and repair once the Bottom-founded Offshore Wind Turbine is in service and to those areas susceptible to corrosion-accelerated fatigue.

For structural members and connections that require a detailed assessment of cumulative fatigue damage, the calculated fatigue life is not to be less than the design life of the Bottom-founded Offshore Wind Turbine times the safety factors for fatigue life [i.e., fatigue design factors (FDFs)] as specified in 5-2/13 TABLE 2.

Fatigue resistance of structural details is to be evaluated in accordance with the *ABS Guide for Fatigue Assessment of Offshore Structures*. Suitable S-N curves are to be selected, with special attention being given to the application and limitation of those curves.

The loading conditions for the fatigue assessment are to be in accordance with 5-2/5. A minimum set of Design Load Cases (DLCs) for the fatigue assessment is specified in 4-2/3 TABLE 1 and, if applicable, 4-2/3 TABLE 2, where 'F' in the column titled 'Type of Analysis' designates the fatigue assessment. Loading history of the substructure during transport and installation phases is to be accounted for. Fatigue analyses are to be carried out using an appropriate loading spectrum or time series in accordance with the accepted theories in calculating accumulated damage.

In the case that the design is based on the LRFD approach, the load factors for all load categories are to be taken as 1.0 in the fatigue assessment.

TABLE 2
Safety Factors for Fatigue Life of Structures (Fatigue Design Factors [FDF]) (15 January 2013)

<i>Importance</i>	<i>Inspectable and Repairable</i>	
	<i>Yes</i>	<i>No</i>
Non-Critical	1	3
Critical	3	5

Notes:

- 1 “Critical” indicates that failure of these structural items would result in the rapid loss of structural integrity and produce an event of unacceptable consequence.
- 2 A Fatigue Design Factor of 1.0 is applicable to
 - Inspectable and repairable non-critical structural members above the splash zone
 - Diver or ROV inspectable and repairable redundant framing
- 3 (15 January 2013) For critical structure members and the turbine tower structure installed above the splash zone, a Fatigue Design Factor of 2.0 may be applied provided that these structures can be inspected during periodic survey or when structural damage is suspected such that critical crack development can be detected and repaired.

15 Stresses in Connections

Connections of structural members are to be developed to verify effective load transmission between joined members, to minimize stress concentration and to prevent excessive punching shear. Connection details are also to be designed to minimize undue constraints against overall ductile behavior and to minimize the effects of post-weld shrinkage. Undue concentration of welding is to be avoided.

The design of tubular joints may be in accordance with the relevant requirements in API RP 2A.

17 Structure-Pile Connections (1 July 2020)

The connection of the substructure of an offshore wind turbine to the foundation and to the turbine tower is to be accomplished by positive, controlled means such as welding, grouting, or other mechanical connectors. Such connections are to be capable of withstanding anticipated static, dynamic, and long-term cyclic loadings. Details of mechanical connectors are to be submitted for review.

In the design of a grouted connection, consideration is to be given to the use of mechanical shear connectors as their presence increases the strength of the connection and alleviates the effect of **long-term** grout shrinkage. Adequate clearance in the annulus is to be provided for proper placement of the grout. Reliable means for the introduction of the grout to the annulus are to be provided in order to achieve complete filling of the annulus and to minimize the possibility of dilution of the grout and the **creation** of voids in the grout. Wipers or similar devices are to be used to minimize intrusion of mud into the annulus between the pile and the substructure leg during installation.

Particulars of grouting mixtures used in the grouted pile connections are to be submitted for review.

For grouted connections undertaking axial loads, the design guidance can be found in API RP 2A. Special attention is to be paid to the limitation of geometric configurations imposed by API RP 2A.

For those grouted pile connections expected to undertake bending moments, their strengths are to be assessed by suitable analysis methods or by tests. The assessment results are to be submitted for review.

For grouted connections whose geometries are not covered by the existing design criteria, special consideration is to be given to the effects of reduced confinement on allowable bond stress, and suitable analyses or tests are to be submitted for review by ABS.

For bolted flange connections, special care is to be taken to verify evenness of contact surface to avoid overstressing of bolts. The design and installation are to be in accordance with recognized standards such as the *AISC Steel Construction Manual*. Consideration is to be given to friction factors, relaxation, stress corrosion cracking, bolt fatigue, brittle failure, and other factors or combinations that may be present.

The allowable stresses to be employed in the design of foundation structure for steel gravity bases or piles are to be in accordance with 5-2/9.3. In the case that the design is based on the LRFD approach, the load and resistance factors to be employed in the design of structure-pile connections are to be in accordance with 5-2/9.5. The design of laterally loaded piles is in accordance with 5-4/9.

19 Structural Response to Hydrostatic Loads

Analyses of the structural stability are to be performed to demonstrate the ability of structural parts to withstand hydrostatic collapse at the expected water depth. Hydrostatic collapse is to be checked in accordance with the *ABS Guide for Buckling and Ultimate Strength Assessment for Offshore Structures*.

21 Deflections (1 July 2020)

The substructure deflections that may affect proper operations of the RNA, the design of piles, and other structures in way of the substructure are to be considered. Where appropriate, the associated geometric nonlinearity is to be considered.

23 Local Structures (1 July 2020)

Structures that do not directly contribute to the overall strength of the substructure, i.e., their loss or damage does not impair the structural integrity of the substructure, are considered to be local structures.

Local structures including the interface with the power cable system (I-tube and J-tube support structure and foundation) are to be adequate for the nature and magnitude of applied loads. The criteria of 5-2/9 apply in the design of local structural components, except for those structural parts whose primary function is to absorb energy, in which case sufficient ductility is to be demonstrated.

CHAPTER 5 Structure and Foundation Design

SECTION 3 Concrete Structures

1 General (1 July 2020)

The requirements of this Section are to be applied to the **substructure** constructed of reinforced and prestressed concrete.

The contents of Section 5-1 are to be consulted regarding general design requirements.

1.1 Materials (1 July 2020)

Unless otherwise specified, the requirements of this Section are intended for the **substructure** constructed of materials manufactured and having properties as specified in Section 2-1. Use of materials having properties differing from those specified in Section 2-1 will be specially considered by ABS. Specifications for alternative materials, details of the proposed methods of manufacture and, where available, evidence of satisfactory previous performance, are to be submitted for review.

For structural lightweight concrete, the reference is made to ACI 213R, and lightweight aggregates are to conform to the requirements of ASTM C330.

1.3 Durability

Materials, concrete mix proportions, construction procedures and quality control are to be chosen to produce satisfactory durability for structures located in a marine environment. Issues to be specifically addressed include chemical deterioration of concrete, corrosion of the reinforcement and hardware, abrasion of concrete, freeze-thaw durability, and fire hazards as they pertain to the zones of exposure defined in 5-1/5.11.

Test mixes are to be prepared and tested early in the design phase to verify that proper values of strength, creep, alkali resistance, etc. can be achieved.

1.5 Access for Inspection

The components of the structure are to be designed to enable their inspection during construction and, to the extent practicable, periodic survey after installation.

3 Design Method (1 July 2020)

The criteria specified in this Section for concrete structures are based on the Load and Resistance Factor Design (LRFD) approach.

Where the Survival Load Cases in accordance with 4-2/5 are assessed, all partial factors are to be taken as 1.0 and structural members can reach their ultimate capacities or beyond provided the global strength of the substructure is not compromised.

3.1 Load Magnitude

The magnitude of a design load for a given type of loading k is obtained by multiplying the load, F_k , by the appropriate load factor, c_k (i.e., design load = $c_k F_k$).

3.3 Design Strength

In the analysis of sections, the design strength of a given material is obtained by multiplying the material strength, f_k , by the appropriate strength reduction factor, ϕ (i.e., design strength = ϕf_k). The material strength, f_k , for concrete is the specified compression strength of concrete (f'_c) after 28 days and for steel is the minimum specified yield strength (f_y). See also 5-3/7.3.

3.5 Design Reference (1 July 2020)

Design considerations for concrete substructure not directly addressed in this Guide are to follow the requirements of the ACI 318 and ACI 357, or equivalent.

5 Design Requirements

5.1 General (1 July 2020)

The strength of the concrete substructure of an offshore wind turbine is to be such that adequate safety exists against failure of the structure or its components. Among the modes of possible failure to be considered are the following:

- i) Loss of overall equilibrium
- ii) Failure of critical section
- iii) Instability resulting from large deformation
- iv) Excessive plastic or creep deformation

The serviceability of the substructure is to be assessed. The following items are to be considered in relation to their potential influences on the serviceability of the structure.

- i) Cracking and spalling
- ii) Deformation
- iii) Corrosion of reinforcement or deterioration of concrete
- iv) Vibration
- v) Leakage

5.3 Load Combinations (1 July 2020)

The load combination is to be in accordance with 5-1/5.1 in general. The load categories referred in this Section (i.e., dead loads, live loads, deformation loads, and environmental loads) are defined in 4-1/3.

The Design Load Cases (DLCs) in 4-2/3 TABLE 1 and 4-2/Table 3, if applicable, are to be assessed as a minimum requirement of design load conditions. DLCs for ice conditions are to be considered in accordance with 4-2/3 for the installation site where ice is expected to occur.

For the design of the concrete substructure of an offshore wind turbine, the partial safety factors for loads are to follow 5-2/9.5.

While the critical design loadings are to be identified from the load combinations given above, the other simultaneously occurring load combinations during construction, transport and installation phases are to be considered if they can cause critical load effects.

5.5 Strength Reduction Factors

The strength of a member or a cross section is to be calculated in accordance with the provisions of 5-3/7 and it is to be multiplied by the following strength reduction factor, ϕ .

- i) In the case of bending without axial tension, $\phi = 0.90$
- ii) In the case of axial compression or axial compression combined with bending.
 - For reinforced members with spiral reinforcement, $\phi = 0.70$
 - For other reinforced members (excluding slabs and shells), $\phi = 0.65$

The values given in the above for two types of members may be increased linearly to 0.9 as ϕP_u decreases from $0.1f'_c A_g$ or ϕP_b , whichever is smaller, to zero, where

f'_c = specified compression strength of concrete

A_g = gross area of section

P_u = axial design load in compression member

P_b = axial load capacity assuming simultaneous occurrence of the ultimate strain of concrete and yielding of tension steel

- For slabs and shells, $\phi = 0.70$
- iii) In the case of shear and torsion, $\phi = 0.75$
- iv) In the case of bearing on concrete, $\phi = 0.65$, except for post-tensioning anchorage bearing. For bearing on concrete in post-tension anchorage, $\phi = 0.85$.

Alternatively, the expected strength of concrete members can be determined by using idealized stress-strain curves and material factors (c_M) given in ACI 357R. The material factors applied to the stress-strain curves limit the maximum stress to achieve the desired reliability similar to using the strength reduction factors given above. The strength reduction factors (ϕ) and the material factors (c_M) are not to be used simultaneously.

5.7 Fatigue (1 July 2020)

The fatigue strength of the concrete substructure of an offshore wind turbine is considered satisfactory if under the unfactored fatigue loads (i.e., $c_k = 1$) the following conditions are satisfied. The fatigue analysis is based on the stress in a critical section. The stress range is to be derived from all fatigue load cycles with a 1-year return probability level.

- i) The stress range in reinforcing or prestressing steel does not exceed 138 MPa (20 ksi), or where reinforcement is bent, welded or spliced, 69 MPa (10 ksi).
- ii) There is no membrane tensile stress in concrete and not more than 1.4 MPa (200 psi) flexural tensile stress in concrete.
- iii) The stress range in compression in concrete does not exceed $0.5f'_c$ where f'_c is the specified compressive strength of concrete.
- iv) Where maximum shear exceeds the allowable shear of the concrete alone, and where the cyclic range is more than half the maximum allowable shear in the concrete alone, all shear is taken by reinforcement. In determining the allowable shear of the concrete alone, the influence of permanent compressive stress may be taken into account.

- v) In situations where fatigue stress ranges allow greater latitude than those under the serviceability requirements given in 5-3/5.9.2 TABLE 1, the latter condition is to assume precedence.
- vi) Bond stress does not exceed 50% of that permitted for static loads. If lap splices of reinforcement or pretensioning anchorage development are subject to cyclic tensile stresses greater than 50% of the allowable static stress, the lap length or prestressing development length is to be increased by 50%.

Where the above nominal values are exceeded, an in-depth fatigue analysis is to be performed. In such an analysis, the possible reduction of material strength is to be taken into account on the basis of appropriate data (S-N curves) corresponding to the 95th percentile of specimen survival. In this regard, consideration is to be given not only to the effects of fatigue induced by normal stresses, but also to fatigue effects due to shear and bond stresses under unfactored load combinations.

Particular attention is to be given to submerged areas subjected to the low-cycle, high-stress components of the loading history.

In prestressed members containing unbonded reinforcement, special attention is to be given to the possibility of fatigue in the anchorages or couplers that may be subjected to corrosive action.

Where a fatigue analysis is performed for the concrete **substructure**, the safety factors for fatigue life [i.e., fatigue design factors (FDFs)] are to be in accordance with 5-2/13 TABLE 2, except that the calculated fatigue life is to be at least twice the design life of the Bottom-founded Offshore Wind Turbine. In order to estimate the cumulative fatigue damage under variable amplitude stresses, a recognized cumulative rule is to be used. Miner's rule is an acceptable method for the cumulative fatigue damage analysis.

5.9 Serviceability Requirements

5.9.1 Serviceability (1 July 2020)

The serviceability of the concrete **substructure** is to be checked by the use of stress-strain diagrams, as depicted in 5-3/Figure 1 and 5-3/Figure 2. The strength reduction factor, ϕ , and partial safety factors for loads, c_k , are to be taken as 1.0. The unfactored ($c_k = 1.0$) load combination of most unfavorable dead loads, deformation loads and live loads as well as the design environmental loads is to be applied.

Using this method, the reinforcing stresses are to be limited in compliance with 5-3/5.9.2 TABLE 1. Additionally for hollow structural cross sections, the maximum permissible membrane strain across the walls is not to cause cracking under any combination of unfactored loads. For structures prestressed in one direction only, tensile stresses in reinforcement transverse to the prestressing steel are to be limited so that the strains at the plane of the prestressing steel do not exceed D_{ps}/E_s , where D_{ps} is as defined in 5-3/5.9.2 TABLE 1 and E_s is the modulus of elasticity of reinforcement (see 5-3/7.3).

Alternative criteria such as those which directly limit crack width may also be considered.

5.9.2 Liquid-Containing Structures

The following criteria are to be satisfied for liquid-containing structures to verify adequate design against leakage.

- i) The reinforcing steel stresses are to be in accordance with section 5-3/5.9.1.
- ii) The compression zone is to extend over 25% of the wall thickness or 205 mm (8 in), whichever is less.
- iii) There is to be no membrane tensile stress unless other construction arrangements are made, such as the use of special barriers to prevent leakage.

TABLE 1
Allowable Tensile Stresses for Prestress and Reinforcing Steel to Control Cracking

Stage	Loading	Allowable Stress, MPa (ksi)	
		Reinforcing Steel, f_s	Prestressing Tendons, D_{ps}
Construction: where cracking during construction would be detrimental to the completed structure	All loads on the structure during construction	160 (23.0)	130 (18.5)
Construction: where cracking during construction is not detrimental to the completed structure	All loads on the structure during construction	210 (30.0) or $0.6 f_y$, whichever is less	130 (18.5)
Transportation and installation	All loads on the structure during transportation and installation	160 (23.0)	130 (18.5)
At offshore site	Dead and live loads plus design environmental loads	$0.8 f_y$	
f_y = yield stress of the reinforcing steel f_s = allowable stress in the reinforcing steel D_{ps} = increase in tensile stress in prestressing steel with reference to the stress at zero strain in the concrete.			

7 Analysis and Design

7.1 General (1 July 2020)

Generally, the analysis of a concrete structure may be performed under the assumption of linearly elastic materials and linearly elastic structural behavior in accordance with the requirements of ACI 318 and the additional requirements of this Subsection. The material properties to be used in analysis are to conform to 5-3/7.3. However, the inelastic behavior of concrete based on the true variation of the modulus of elasticity with stress and the geometric nonlinearities, including the effects of initial deviation of the structure from the design geometry, are to be taken into account whenever their effects reduce the strength of the **substructure**. The beneficial effects of the concrete's nonlinear behavior may be accounted for in the analysis and design of the structure to resist dynamic loadings.

Where required, the dynamic behavior of a concrete structure may be investigated using a linear structural model, but soil-structural impedances are to be suitably taken into account. The analysis of the structure under earthquake conditions may be performed under the assumption of elasto-plastic behavior due to yielding, provided the requirements of 5-3/7.13 are satisfied.

7.3 Material Properties for Structural Analysis

7.3.1 Specified Compressive Strength

The specified compressive strength of concrete, f'_c , is to be based on 28-day tests performed in accordance with specifications ASTM C172, ASTM C31 and ASTM C39.

7.3.2 Early Loadings

For structures that are subjected to loadings before the end of the 28-day hardening period of concrete, the compressive strength of concrete is to be taken at the actual age of concrete at the time of loading.

7.3.3 Early Strength –Concrete (1 July 2020)

For early-strength concrete, the age for the tests for f'_c may be determined based on the cement manufacturer's certificate.

7.3.4 Modulus of Elasticity – Concrete

For the purposes of structural analyses and deflection checks, the modulus of elasticity, E_c , of normal weight concrete may be assumed as equal to $4733(f'_c)^{0.5}$ MPa ($57(f'_c)^{0.5}$ ksi), or determined from stress-strain curves developed by tests (see 5-3/7.11.2 FIGURE 1). When the latter method is used, the modulus of elasticity is to be determined using the secant modulus for the stress equal to $0.50 f'_c$.

7.3.5 Uniaxial Compression – Concrete

In lieu of tests, the stress-strain relation shown in 5-3/7.11.2 FIGURE 1 may be used for uniaxial compression of concrete.

7.3.6 Poisson Ratio

The Poisson ratio of concrete may be taken equal to 0.20.

7.3.7 Modulus of Elasticity – Reinforcement

The modulus of elasticity, E_s of non-prestressed steel reinforcement is to be taken as 200×10^3 MPa (29×10^3 ksi). The modulus of elasticity of prestressing tendons is to be determined by tests.

7.3.8 Uniaxial Tension – Reinforcement

The stress-strain relation of non-prestressed steel reinforcement in uniaxial tension is to be assumed as shown in 5-3/7.11.2 FIGURE 2. The stress-strain relation of prestressing tendons is to be determined by tests, or taken from the manufacturer's certificate.

7.3.9 Yield Strength – Reinforcement

If the specified yield strength, f_y , of non-prestressed reinforcement exceeds 420 MPa (60 ksi), the value of f_y used in the analysis is to be taken as the stress corresponding to a strain of 0.35%.

7.5 Analysis of Plates, Shells, and Folded Plates (1 July 2020)

In all analyses of shell structures, the theory employed in analysis is not to be based solely on membrane or direct stress approaches. The buckling strength of plate and shell structures is to be checked by an analysis that takes into account the geometrical imperfections of the structure, the inelastic behavior of concrete and the creep deformations of concrete under sustained loading. Special attention is to be given to structures subjected to external pressure and the possibility of their collapse (implosion) by failure of concrete in compression.

7.7 Deflection Analysis

Immediate deflections may be determined by the methods of linear structural analysis. For the purposes of deflection analysis, the member stiffness is to be computed using the material properties specified in the design and is to take into account the effect of cracks in tension zones of concrete. The effect of creep strain in concrete is to be taken into account in the computations of deflections under sustained loadings.

7.9 Analysis and Design for Shear and Torsion (1 July 2020)

The analysis and design of members subjected to shear or torsion or to combined shear and torsion is to adhere with the applicable requirements of ACI 318 or their equivalent.

7.11 Analysis and Design for Bending and Axial Loads

7.11.1 Assumed Conditions

The analysis and design of members subjected to bending and axial loads are to be based on the following assumptions:

- i) The strains in steel and concrete are proportional to the distance from the neutral axis.
- ii) Tensile strength of the concrete is to be neglected, except in prestressed concrete members under unfactored loads, where the requirements in 5-3/5.9 apply.
- iii) The stress in steel is to be taken as equal to E_s (see 5-3/7.3.7) times the steel strain, but not larger than f_y (see 5-3/7.3.9).
- iv) The stresses in the compression zone of concrete are to be assumed to vary with strain according to the curve given in 5-3/7.11.2 FIGURE 1 or any other conservative rule. Rectangular distribution of compressive stresses in concrete specified by ACI 318 may be used.
- v) The maximum strain in concrete at the ultimate state is not to be larger than 0.30%.

7.11.2 Failure

The members in bending are to be designed in such a way that any section yielding of steel occurs prior to compressive failure of concrete.

FIGURE 1
Idealized Stress-Strain Relation for Concrete in Uniaxial Compression

E_c is defined in 5-3/7.3

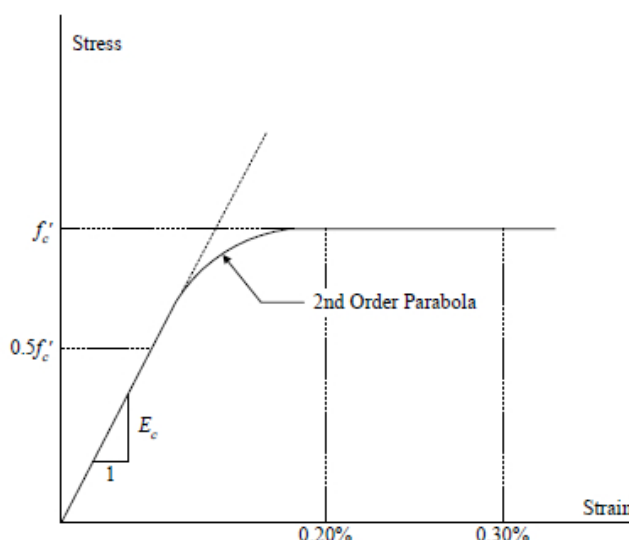
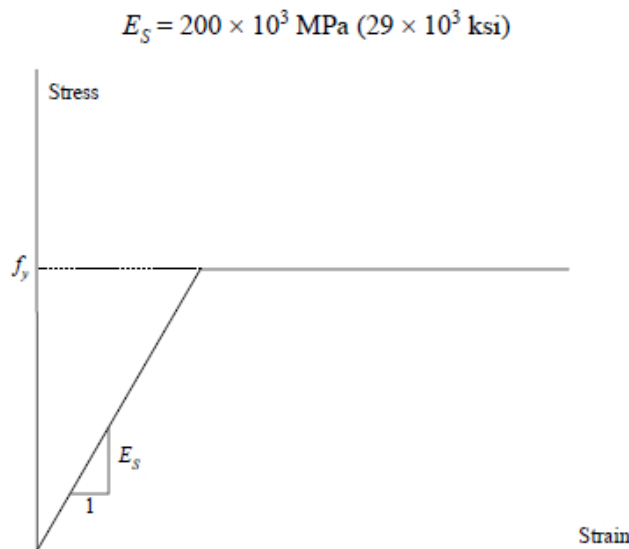


FIGURE 2
Idealized Stress-Strain Relation for Non-Prestressed Steel in Uniaxial Tension



7.13 Seismic Analysis

7.13.1 Dynamic Analysis (1 July 2020)

For the **substructure** to be located at sites known to be seismically active (see 5-3/7.15), dynamic analyses are to be performed to determine the response of the structure to design earthquake loading. The **substructure** is to be designed to withstand this loading without damage. In addition, a ductility check is also to be performed to verify that the structure has sufficient ductility to resist deflections more severe than those resulting from the design earthquake loading without the collapse of the **substructure** or any major structural component. Reference is made to 3-6/9 and 4-3/11 for the Strength Level and Ductile Level earthquakes as well as the earthquake loads.

7.13.2 Design Conditions

The dynamic analysis for earthquake loadings is to be performed with consideration of:

- i) The interaction of all load bearing or load carrying components of the structure
- ii) The compliance of the soil and the dynamic soil-structure interaction
- iii) The dynamic effects of the ambient and contained fluids

7.13.3 Method of Analysis (15 January 2013)

The dynamic analysis for earthquake loadings may be performed by any recognized method, such as direct integration of the equations of motion or the response spectra method.

7.13.4 Ductility Check

In the ductility check, ground motions (e.g., spectral ordinates) at least twice those used for the design earthquake are to be assumed. If the ductility check is performed with the assumption of elasto-plastic behavior of the structure, the selected method of analysis is to be capable of taking into account the non-linearity of the structural model. The possibility of dynamic instability (dynamic buckling) of individual members and of the whole structure is to be considered.

7.15 Seismic Design

7.15.1 Compressive Strain

The compressive strain in concrete at critical sections (including plastic hinge locations) is to be limited to 0.30%, except when greater strain may be accommodated by confining steel.

7.15.2 Flexural Bending or Load Reversals

For structural members or sections subject to flexural bending or to load reversals, where the percentage of tensile reinforcement exceeds 70% of the reinforcement at which yield stress in the steel is reached simultaneously with compression failure in the concrete, special confining reinforcement (e.g., T-headed bars) and/or compressive reinforcement are to be provided to prevent brittle failure in the compressive zone of concrete.

7.15.3 Web Reinforcement

Web reinforcement (stirrups) of flexural members is to be designed for shear forces which develop at full plastic bending capacity of end sections. In addition:

- i)* The diameter of rods used as stirrups is not to be less than 10 mm (#3 bar)
- ii)* Only closed stirrups (stirrup ties) are to be used. T-headed bars or other mechanically headed bars may be used if their effectiveness has been verified.
- iii)* The spacing of stirrups is not to exceed the lesser of $d/2$ or 16 bar diameters of compressive reinforcement, where d is the distance from the extreme compression fiber to the centroid of tensile reinforcement. Tails of stirrups are to be anchored within a confined zone (i.e., turned inward).

7.15.4 Splices

No splice is allowed within a distance d , defined in 5-3/7.15.3, from a plastic hinge. Lap splices are to be designed in accordance with ACI 318. Mechanical and welded splices are permitted. Mechanical splices are to be in compliance with ACI 349.

9 Design Details

9.1 Concrete Cover

9.1.1 General

The following minimum concrete cover for reinforcing bars is required:

- i)* Atmospheric zone not subject to salt spray: 50 mm (2 in.)
- ii)* Splash and atmospheric zones subjected to salt spray and exposed to soil: 65 mm (2.5 in.)
- iii)* Submerged zone: 50 mm (2 in.)
- iv)* Areas not exposed to weather or soil: 40 mm (1.5 in.)
- v)* Cover of stirrups may be 13 mm (0.5 in.) less than covers listed above

9.1.2 Tendons and Ducts

The concrete cover of prestressing tendons and post-tensioning ducts is to be increased by 25 mm (1 in.) above the values listed in 5-3/9.1.1.

9.1.3 Sections Less Than 500 mm (20 in.) Thick

In sections less than 500 mm (20 in.) thick, the concrete cover of reinforcing bars and stirrups may be reduced below the values listed in 5-3/9.1.1. However, the cover is not to be less than the following:

- i)* 1.5 times the nominal aggregate size
- ii)* 1.5 times the maximum diameter of reinforcement, or 19 mm (0.75 in.)

- iii) Tendons and post-tensioning duct covers are to have 12.5 mm (0.5 in.) added to the above

9.3 Minimum Reinforcement

The minimum requirements of ACI 318 are to be satisfied. In addition, for loadings during all phases of construction, transportation, and operation (including design environmental loading) where tensile stresses occur on a face of the structure, the following minimum reinforcement is to be provided.

$$A_s = (f_t/f_y)bd_e$$

where

- A_s = total cross-section area of reinforcement
- f_t = mean tensile strength of concrete
- f_y = yield stress of the reinforcing steel
- b = width of structural element
- d_e = effective tension zone, to be taken as $1.5c + 10d_b$
- c = cover of reinforcement
- d_b = diameter of reinforcement bar

d_e is to be at least 0.2 times the depth of the section, but not greater than $0.5(h - x)$, where x is the depth of the compression zone prior to cracking and h is the section thickness.

At intersections between structural elements, where transfer of shear forces is essential to the integrity of the structure, adequate transverse reinforcement is to be provided.

9.5 Reinforcement Details

Generally, lapped joints and mechanical splices are to be avoided in structural members subject to significant fatigue loading. Where lapped splices are used in members subject to fatigue, the development length of reinforcing bars is to be twice that required by ACI 318, and lapped bars are to be tied with tie wire. Where mechanical splices are used in members subject to fatigue, the coupled assembly of reinforcing bars and the mechanical coupler are to demonstrate adequate fatigue resistance by test.

Where lapped bars are expected to be subject to tension during operation, through-slab confinement reinforcement is to be considered at the splices. Where longitudinal bars are subject to tension during operation, special consideration is to be given to number of reinforcement with splices at a single location.

Reinforcing steel is to comply with the chemical composition specifications of ACI 359 if welded splices are used.

For anchorage of shear and main reinforcement, mechanically-headed bars (T-headed bars) may be used if their effectiveness has been verified by static and dynamic testing. Shear reinforcement is to be full length without splices. Entire close-up stirrups are to be anchored by hooks or bends of at least 90 degrees followed by a straight leg length of a minimum 12 bar diameters.

9.7 Post Tensioning Ducts

Ducting for post-tensioning ducts may be rigid steel or plastic (polyethylene or polystyrene). Steel tubing is to have a minimum wall thickness of 1 mm. Plastic tubing is to have a minimum wall thickness of 2 mm. Ducts may also be semi-rigid steel, spirally wrapped, of minimum thickness of 0.75 mm, and is to be grout-tight. All splices in steel tubes and semi-rigid duct are to be sleeved and the joints sealed with heat-shrink tape. Joints in plastic duct are to be fused or sleeved and sealed.

The inside diameter of ducts is to be at least 6 mm (0.25 in.) larger than the diameter of the post-tensioning tendon to facilitate grout injection.

Flexible ducts are to be used only in special areas where the rigid or semi-rigid duct is impracticable, such as at sharp bends. A mandrel is to be inserted into the ducts to prevent them from dislocating during concreting.

9.9 Post-Tensioning Anchorages and Couplers

Anchorages for unbonded tendons and couplers are to develop the specified ultimate capacity of the tendons without exceeding the anticipated set. Anchorages for bonded tendons are to develop at least 90% of the specified ultimate capacity of the tendons, when tested in an unbonded condition without exceeding anticipated set. However, 100% of the specified ultimate capacity of the tendons is to be developed after the tendons are bonded in the member.

Anchorage and end fittings are to be permanently protected against corrosion. Post-tensioning anchorages are to preferably be recessed in a pocket which is then filled with concrete. The fill is to be mechanically tied to the structure by reinforcements as well as bonded by epoxy or polymer.

Anchor fittings for unbonded tendons are to be capable of transferring to the concrete a load equal to the capacity of the tendon under both static and cyclic loading conditions.

9.11 Embedded Metals in Concrete (1 July 2020)

Consideration is to be given to **prevention of** corrosion of exposed faces of steel embedment. These embedments are to be separated from the reinforcing steel. Effects of dimensional changes due to factors such as prestressing and temperature changes which may result in fractures near embedments may require provisions to prevent deformation.

11 Construction and Detailing

11.1 General (1 July 2020)

Construction methods and workmanship are to follow accepted practices as described in ACI 301, ACI 318, ACI 357 or other relevant standards. Additional requirements relevant to concrete **substructure** of an offshore wind turbine are included below.

11.3 Mixing, Placing, and Curing of Concrete

11.3.1 Mixing

Mixing of concrete is to conform to the requirements of ACI 318 and ASTM C94.

11.3.2 Cold Weather

In cold weather, concreting in air temperatures below 2°C (35°F) is to be carried out only if special precautions are taken to protect the fresh concrete from damage by frost. The temperature of the concrete at the time of placing is to be at least 4°C (40°F) and the concrete is to be maintained at this or a higher temperature until it has reached a strength of at least 5 MPa (700 psi).

Protection and insulation are to be provided to the concrete where necessary. The aggregates and water used in the mix are to be free from snow, ice and frost. The temperature of the fresh concrete may be raised by heating the mixing water or the aggregates or both. Cement is never to be heated, nor is it to be allowed to come into contact with water at a temperature greater than 60°C (140°F).

11.3.3 Hot Weather

During hot weather, proper attention is to be given to ingredients, production methods, handling, placing, protection and curing to prevent excessive concrete temperatures or water evaporation which will impair the required strength or serviceability of the member or structure. The temperature of concrete as placed is not to exceed 30°C (90°F) and the maximum temperature due to heat of hydration is not to exceed 65°C (145°F).

11.3.4 Curing

Special attention is to be paid to the curing of concrete in order to verify maximum durability and to minimize cracking. Concrete is to be cured with fresh water, whenever possible, to keep the concrete surface wet during hardening. Care is to be taken to avoid the rapid lowering of concrete temperatures (thermal shock) caused by applying cold water to hot concrete surfaces.

11.3.5 Sea Water

Sea water is not to be used for curing reinforced or prestressed concrete, although, if demanded by the construction program, “young” concrete may be submerged in sea water provided it has gained sufficient strength to withstand physical damage. When there is doubt about the ability to keep concrete surfaces permanently wet for the whole curing period, a heavy duty membrane curing compound is to be used.

11.3.6 Temperature Rise

The rise of temperature in the concrete, caused by the heat of hydration of the cement, is to be controlled to prevent steep temperature stress gradients which could cause cracking of the concrete. Since the heat of hydration may cause significant expansion, members must be free to contract, so as not to induce excessive cracking. In general, when sections thicker than 610 mm (2 ft) are concreted, the temperature gradients between internal concrete and external ambient conditions are to be kept below 20°C (68°F).

11.3.7 Joints

Construction joints are to be made and located in such a way as not to impair the strength and crack resistance of the structure. Where a joint is to be made, the surface of the concrete is to be thoroughly cleaned and all laitance and standing water removed. Vertical joints are to be thoroughly wetted and coated with neat cement grout or equivalent enriched cement paste or epoxy coating immediately before placing of new concrete.

11.3.8 Watertight Joints

Whenever watertight construction joints are required, in addition to the above provisions, the heavy aggregate of the existing concrete is to be exposed and an epoxide-resin bonding compound is to be sprayed on just before concreting. In this case, the neat cement grout can be omitted.

11.5 Reinforcement

The reinforcement is to be free from loose rust, grease, oil, deposits of salt or any other material likely to affect the durability or bond of the reinforcement. The specified cover to the reinforcement is to be maintained accurately. Special care is to be taken to correctly position and rigidly hold the reinforcement so as to prevent displacement during concreting.

11.7 Prestressing Tendons, Ducts, and Grouting

11.7.1 General

Further guidance on prestressing steels, sheathing, grouts and procedures to be used when storing, making up, positioning, tensioning and grouting tendons can be found in the relevant sections of ACI 318, Prestressed Concrete Institute (PCI) publications, Federation Internationale de la Precontrainte (FIP) Recommended Practices, and the specialist literature.

11.7.2 Cleanliness

All steel for prestressing tendons is to be clean and free from grease, insoluble oil, deposits of salt or any other material likely to affect the durability or bond of the tendons.

11.7.3 Storage

During storage, prestressing tendons are to be kept clear of the ground and protected from weather, moisture from the ground, sea spray and mist. No welding, flame cutting or similar operations are to be carried out on or adjacent to prestressing tendons under any circumstances where the temperature of the tendons could be raised or weld splash could reach them.

11.7.4 Protective Coatings

Where protective wrappings or coatings are used on prestressing tendons, they are to be chemically neutral so as not to produce chemical or electrochemical corrosive attack on the tendons.

11.7.5 Entry of Water (1 July 2020)

All ducts are to be watertight and all splices carefully taped to prevent the ingress of water, grout or concrete. During construction, the ends of ducts are to be capped and sealed to prevent the ingress of sea water. Ducts may be protected from excessive rust by the use of chemically neutral protective agents such as vapor phase inhibitor powder.

11.7.6 Grouting

Where ducts are to be grouted, all oil or similar material used for internal protection of the sheathing is to be removed before grouting. However, water-soluble oil used internally in the ducts or on the tendons may be left on, to be removed by the initial portion of the grout.

11.7.7 Air Vents

Air vents are to be provided at all crests in the duct profile. Threaded grout entries, which permit the use of a screwed connector from the grout pump, may be used with advantage.

11.7.8 Procedures

For long vertical tendons, the grout mixes, admixtures and grouting procedures are to be checked to verify that no water is trapped at the upper end of the tendon due to excessive bleeding or other causes. Suitable admixtures known to have no injurious effects on the metal or concrete may be used for grouting to increase workability and to reduce bleeding and shrinkage. The temperature of members is to be maintained above 10°C (50°F) for at least 48 hours after grouting. General guidance on grouting can be found in the specialist literature. Holes left by unused ducts or by climbing rods of slipforms are to be grouted in the same manner as described above.

CHAPTER 5 Structure and Foundation Design

SECTION 4 Foundations

1 General (1 July 2020)

This Section addresses site investigations, design considerations for the supporting soil, and the influence of the soil and foundation on the Bottom-founded Offshore Wind Turbine. The degree of design conservatism is to reflect prior experience under similar conditions, the manner and extent of data collection, the scatter of design data, and the consequences of failure. For cases where the limits of applicability of any method of calculation employed are not well defined, or where the soil characteristics vary significantly, more than one method of calculation or a parametric study of the sensitivity of the relevant design data is to be used.

If the design involves novel foundations or unique soil conditions, model testing may be required to verify design assumptions and methodologies.

3 Documentation (1 July 2020)

The design data and documentation to be submitted for ABS review are summarized in 1-1/9. As a minimum, the following documents are to be submitted, as applicable.

- i) Site Investigation reports including geological survey, geophysical/geohazard survey and geotechnical investigation and reports
- ii) A **site-specific** seismic hazard report if the reference site is within a seismic zone
- iii) As applicable, the results of studies to assess the following effects are also to be submitted. In these studies, the structure is to be considered present
 - Scouring potential of the sea floor
 - Hydraulic instability and the occurrence of sand waves
 - Instability of slopes in the area where the Support Structure is to be installed
 - Liquefaction and other soil instabilities
 - Geo-hazards such as faults, gas seeps etc
 - For Arctic areas, possible degradation of subsea permafrost layers
 - Soils conditions in the vicinity of footprints left by temporarily situated service units or other service units

- Effects of volcanic sands, organic matter, carbonate soil, calcareous sands and other substances which degrade the strength of the soil foundation
- iv) Design reports covering the following:
 - Holding capacity of the foundation
 - Foundation structure design
 - Fatigue analysis if applicable
 - Installation analyses
 - Corrosion protection
- v) Design drawings and specifications for material, welding and fabrication
- vi) Foundation installation procedure

5 Site Investigation

5.1 General (1 July 2020)

The site investigation program is to consist of the following three phases.

- i) Sea Floor Survey to obtain relevant geophysical data
- ii) Geological Survey to obtain data of a regional nature concerning the site
- iii) Subsurface Investigation and Testing to obtain the necessary geotechnical data

The results of these investigations are to be the bases for the additional site related studies which are listed in 5-4/3, item *iii*) above.

A complete site investigation program is to be accomplished. However, use of the complete or partial results of a previously completed site investigation as the design basis for another similarly designed and adjacent structure is permitted when the adequacy of the previous site's investigation for the new location is satisfactorily demonstrated.

When deciding the area to be investigated, due allowance is to be given to the accuracy of positioning devices used on the vessel employed in the site investigation such that the data obtained are pertinent to the actual location of the Bottom-founded Offshore Wind Turbine.

5.3 Sea Floor Survey (1 July 2020)

Geophysical data for the conditions existing at and near the surface of the sea floor are to be obtained. The following information is to be obtained where applicable to the planned Bottom-founded Offshore Wind Turbine.

- i) Soundings or contours of the sea bed
- ii) Position of bottom shapes which might affect scour
- iii) The presence of boulders, obstructions, and small craters
- iv) Gas seeps
- v) Shallow faults
- vi) Slump blocks
- vii) Ice scour of sea floor sediments
- viii) Subsea permafrost or ice bonded soils

5.5 Geological Survey (1 July 2020)

Data of the regional geological characteristics which can affect the design and siting of the Bottom-founded Offshore Wind Turbine are to be considered in planning the subsurface investigation, and they are also to be used to verify that the findings of the subsurface investigation are consistent with known geological conditions.

Where necessary, an assessment of the seismic activity at the site is to be made. Particular emphasis is to be placed on the identification of fault zones, the extent and geometry of faulting and attenuation effects due to conditions in the region of the site.

5.7 Subsurface Investigation

The subsurface investigation is to obtain reliable geotechnical data concerning the stratigraphy and properties of the soil. These data are to be used to assess whether the desired level of structural safety and performance can be obtained and to assess the feasibility of the proposed method of installation.

Consistent with the stated objective, the subsurface investigation program is to consist of an adequate number of in-situ tests, borings, probings and geophysical investigation to examine all important soil and rock strata at each foundation location. The extent of the investigation is to be discussed and agreed upon with ABS prior to the beginning of the subsurface investigation and may be adjusted based on the results of the Sea Floor Survey and Geological survey.

Soil data are to be taken in the vicinity of the foundation locations, including seabed samples for evaluation of scour potential. An interpretation of such data is to be submitted by a recognized geotechnical consultant. To establish the soil characteristics at the foundation locations, borings or probings are to be taken at all foundation locations to a suitable depth of at least the anticipated depth of any pile penetrations plus a consideration for the soil variability as described below. If probings are used, they are to be calibrated with the data obtained from conventional boring in the vicinity of the probing.

As an alternative, geophysical surveys may be carried out and correlated with at least two borings or probings in the vicinity to establish the soil profile at each foundation location. The integration of geophysical, geotechnical and geological findings are to be carried out by recognized consultants. If the foundation location is at a significant distance from the boring/probing location, additional number of verification borings/probings may be required to validate the extrapolated data. To account for possible soil variability, a lower bound and an upper bound soil profile are to be established for use in pile holding capacity calculations and installation analyses, respectively.

For pile-supported structures, the minimum depth of a bore hole, for either individual or clustered piles, is to be the design penetration plus a zone of influence. The zone of influence is to be at least 15.2 m (50 ft) or 1.5 times the diameter of the cluster, whichever is greater, unless it can be demonstrated by analytical methods that a lesser depth is justified. Additional bore holes of lesser depth are required if discontinuities in the soil are likely to exist within the area of the structure.

For a gravity-type foundation, the required depth of at least one boring is to be at least equal to the larger horizontal dimension or three times the smaller dimension of the base, whichever is greater. In-situ tests are to be carried out, where possible, to a depth that will include the anticipated shearing failure zone.

A reasonably continuous profile is to be obtained during recovery of the boring samples. The recovery of the materials to a depth of 12 m (40 ft) below the mudline is to be as complete as possible. Thereafter, samples at significant changes in strata are to be obtained, at approximately 3 m (10 ft) intervals to 61 m (200 ft) and approximately 8 m (25 ft) intervals below 61 m (200 ft).

The existence of carbonate soils is to be determined in frontier areas or areas known to contain carbonate materials during the subsurface investigation. Additional field and laboratory testing may be required when a soil profile shows the contained carbonate materials are more than 15% of the soil fraction.

5.9 Soil Testing Program

The testing program is to reveal the necessary engineering properties of the soil including strength, classification and deformation properties of the soil. Testing is to be performed in accordance with recognized standards.

At least one undrained strength test (vane, drop cone, unconfined compression, etc.) on selected recovered cohesive samples is to be performed in the field.

Where practicable, a standard penetration test or equivalent on each significant sand stratum is to be performed, recovering samples where possible. Field samples for laboratory work are to be retained and carefully packaged to minimize changes in moisture content and disturbance.

Samples from the field are to be sent to a recognized laboratory for further testing. They are to be accurately labeled and the results of visual inspection recorded. The testing in the laboratory is to include at least the following.

- i) Perform unconfined compression tests on clay strata where needed to supplement field data.
- ii) Determine water content and Atterberg limits on selected cohesive samples.
- iii) Determine density of selected samples.
- iv) As necessary, develop appropriate constitutive parameters or stress-strain relationships from either unconfined compression tests, unconsolidated undrained triaxial compression tests, or consolidated undrained triaxial compression tests.
- v) Perform grain size sieve analysis, complete with percentage passing 200 sieve, on each significant sand or silt stratum.
- vi) Other advanced testing to determine soil parameters necessary for detailed geotechnical analyses.

For pile-supported structures, consideration is also to be given to the need for additional tests to adequately describe the dynamic, creep and set-up characteristics of the soil as well as the static and cyclic lateral properties of soil-pile system.

For gravity structures, laboratory tests are also to include, where necessary, the following.

- i) Shear strength tests with pore pressure measurements. The shear strength parameters and pore-water pressures are to be measured for the relevant stress conditions
- ii) Cyclic loading tests with deformation and pore pressure measurements to determine the soil behavior during alternating stress
- iii) Permeability and consolidation tests performed as required

7 Foundation Design Requirements

7.1 General (1 July 2020)

The loadings used in the design of foundations are to include those defined in 5-4/7.13 and those experienced during installation. Foundation displacements are to be evaluated to the extent necessary to conclude that they are within the limits that do not impair the intended function and safety of the Bottom-founded Offshore Wind Turbine.

The soil and the offshore wind turbine are to be considered as an interactive system. The results of analyses, as required in subsequent paragraphs, are to be evaluated from this point of view.

In addition to this Subsection, specific design requirements are described in 5-4/9 for pile foundations and 5-4/11 for gravity structures. Other types of foundations are subject to special considerations by ABS.

Where the Survival Load Cases in accordance with 4-2/5 are assessed, the ultimate strength analysis method provided in API RP 2SIM is to be applied for the foundation design, where applicable. For the pile foundation, if a global collapse mechanism in the foundation controls the ultimate strength of the Bottom-founded Offshore Wind Turbine, then a geotechnical engineer familiar with assessment should be involved in the assessment. Refer to API RP 2SIM for guidance.

7.3 Cyclic Loading Effects

The influence of cyclic loading on soil properties is to be considered and possible reduction of soil strength is to be investigated and employed in the design of the foundation. In particular, the effects of wind and wave induced forces on the soil properties under the following conditions are to be considered.

- i) Design storm during the initial consolidation phase.
- ii) Short-term effects of the design storm.
- iii) Long-term cumulative effects of several storms, including the design storm, and turbine operations

Reduced soil strength characteristics resulting from these conditions are to be employed in the design.

In seismically active zones, similar deteriorating effects due to repeated loadings are to be considered.

Other possible cyclic load effects, such as changes in load-deflection characteristics, liquefaction potential and slope stability are also to be considered when they are expected to affect the design.

7.5 Scour

Where scour is expected to occur, either effective protection is to be furnished soon after the installation of pile or gravity base, or the depth and lateral extent of scouring, as evaluated in the site investigation program, are to be accounted for in the design of the foundation.

7.7 Deflections and Rotations (1 July 2020)

Tolerable limits of deflections and rotations are to be established based on the type of the substructure and the effects of those movements on the turbine operations. Maximum allowable values of pile or gravity base movements, as limited by the structural considerations, overall stability of the Support Structure and serviceability limit of the wind turbine, are to be considered in the design of the foundation. Pile installation tolerances are also to be considered in the design.

7.9 Soil Strength

The ultimate strength or stability of soil is to be determined using test results which are compatible with the analysis method selected. In a total stress approach, the total shear strength of the soil obtained from simple tests is used. A total stress approach largely ignores changes in the soil's pore water pressure under varying loads and the drainage conditions at the site. When an effective stress approach is used, effective soil strength parameters and pore water pressures are to be determined from tests which predict in-situ total stresses and pore pressures.

7.11 Dynamic and Impact Considerations

For dynamic and impact loading conditions, a realistic and compatible treatment is to be given to the interactive effects between the soil and structure. Dynamic analyses of the Support Structure of an offshore wind turbine may be accomplished by lumped parameter, foundation impedance functions, or by continuum approaches including the use of finite element methods. Such models are to include consideration of the internal and radiational damping provided by the soil and the effects of soil layering.

Studies of the dynamic response of the structure are to include, where applicable, consideration of the nonlinear and inelastic characteristics of the soil, the possibilities of deteriorating strength and increased or decreased damping due to cyclic soil loading, and the added mass of soil subject to acceleration. Where applicable, the influence of nearby structures is to be included in the analysis.

7.13 Loading Conditions (15 January 2013)

Those loading conditions which produce the most unfavorable effects on the foundation during and after installation are to be taken into account. Post installation loadings to be checked are to include at least those relating to the design load conditions, as specified in Chapter 4, Section 2, combined with dead loads, live loads and deformation loads appropriate to the function and operations of an offshore wind turbine.

For areas with potential seismic activity, the foundation is to be designed for sufficient strength to sustain seismic loads. As appropriate, effects of liquefaction shear failure of soft mud and submarine slides are to be taken into account.

7.15 Loads and Soil Conditions Due to Temporarily Situated Structures (1 July 2020)

Changes in soil conditions due to temporarily situated structures, such as self-elevating wind turbine installation units placed near the Support Structure, are to be assessed. These changes and their influence on the structure are to be incorporated in the foundation design such that the intended function and safety of the Bottom-founded Offshore Wind Turbine are not impaired.

9 Pile Foundations

9.1 General (1 July 2020)

The effects of axial, bending, and lateral loads are to be accounted for in the design of pile foundations. The design of a pile is to reflect the interactive behavior between the soil and the pile and between the pile and other components of the Support Structure. Piles are to be designed to withstand in-place and installation loads. Foundation displacements are to be evaluated to the extent necessary to conclude that they are within limits which do not impair the intended function and safety of the Bottom-founded Offshore Wind Turbine.

The following is to be considered in the evaluations of the pile holding capacity.

- i) Cyclic, creep and soil set-up effects on the soil strength
- ii) Punch through failure in layered soils
- iii) Installation tolerances and possible soil disturbance due to protuberances as applicable
- iv) Potential of soil liquefaction and its impact, if the reference site is within a seismic zone
- v) Pile group effect if applicable
- vi) Bottom instability such as slope stability, hydraulic stability (seepage, piping), scour potential, etc.

Where applicable, the effects of close spacing on the load and deflection characteristics of pile groups are to be determined. The allowable load for a group, both axial and lateral, is not to exceed the sum of the apparent individual pile allowable loads reduced by a suitable factor.

The required load conditions and strength criteria of steel structural members are to be in accordance with Section 5-2. Depending on water depth, anticipated loads and the pile configuration, fatigue analysis may be required.

The adequacy of the corrosion protection of the piles is to be evaluated. A corrosion protection system may be designed in accordance with the recognized industry standards, such as those published by NACE. The design life of the corrosion protection system is to be equal or greater than the design life of the Bottom-founded Offshore Wind Turbine.

Methods of pile installation are to be consistent with the type of soil at the site, and with the installation equipment available. Pile installation is to be carried out and supervised by qualified and experienced personnel, and is to be witnessed by the ABS Surveyor.

9.3 Slender Piles

9.3.1 Axial Loads

The axial capacity of slender piles in compression consists of the skin friction, Q_f developed along the length of the pile, and the end bearing, Q_p at the tip of the pile. The axial capacity of a pile subjected to tension is to be equal to or less than the skin friction alone. Predictions of the various parameters needed to evaluate Q_f and Q_p are to be accomplished using a recognized analytical method, such as that recommended in the API RP 2A, or other methods shown to be more appropriate to the conditions at the site.

When required, the acceptability of any method used to predict the components of pile resistance is to be demonstrated by showing satisfactory performance of the method under conditions similar to those existing at the actual site. The results of dynamic pile driving analysis alone are not to be used to predict the axial load capacity of a pile.

When the ultimate capacities of a pile are evaluated using the API method cited above, the allowable axial pile bearing and pullout loads are to be determined by dividing the ultimate capacities by the factor of safety specified in 5-4/9.3.1 TABLE 1. For the Design Earthquake, the factor of safety is to be specially considered.

TABLE 1
Factor of Safety for Axially Loaded Piles

<i>Loading Condition</i>	<i>Factor of Safety</i>
Normal	2.0
Abnormal	1.5

9.3.2 Lateral Loads

In the evaluation of the pile's behavior under lateral loadings, the combined-load-deflection characteristics of the soil and pile, and of the pile and structure are to be taken into account. The representation of pile's lateral deflection when the pile is subjected to lateral loads is to adequately reflect the deterioration of the lateral bearing capacity when the soil is subjected to cyclic loading.

The description of the lateral load versus deflection characteristics ($p - y$ curves) for the various soil strata is to be based on constitutive data obtained from suitable soil tests. Reference may be made to the API RP 2A for a procedure to evaluate the load-deflection characteristics of laterally loaded piles. However, the use of alternative methods is permitted when they are more appropriate for conditions at the site.

Where applicable, the rapidly deteriorating cyclic bearing capacity of stiff clays, especially those exhibiting the presence of a secondary structure, is to be taken into account.

The strength criteria for steel structural members of the pile subjected to lateral loads are to be in accordance with 5-2/9.

9.5 Large Diameter Piles (1 July 2020)

The design method in 5-4/9.3 slender piles may not be applicable to large diameter piles. Due to the geometry of the pile, the failure modes of the soil may be different than those applicable for slender piles as described in 5-4/9.3. A suitable analytical method, such as the finite element analyses, may be employed to evaluate the holding capacity of large diameter piles and the adequacy of the piles to withstand in-place loads. Soil properties necessary for these analyses are to be established in consultation with the geotechnical consultant. Where the limit equilibrium method is used, different failure modes, which are affected by various factors such as the pile load and load direction, are to be considered to establish the

holding capacity of a pile. Model tests, centrifuge tests, or **full-scale** pile testing may be carried out to provide justification of the design method.

9.7 Pile Installation Analysis (15 January 2013)

Pile installation analyses including lifting, transportation and penetration analyses are to be carried out according to the relevant requirements in API RP 2A.

For driven piles, the pile stick-up and driveability analysis are to be performed to evaluate the adequacy of the pile to withstand loads during driving conditions in accordance with API RP 2A.

Under the stick-up condition, the pile's adequacy to withstand the self-weight, hammer weight and environmental loads acting on the pile section above the mudline, if applicable, is to be verified.

Pile driveability study is to use acceptable methods and appropriate analysis parameters for operating efficiency, hammer data and accessories, etc. Driving stresses, number of blows per unit depth and total blow count required for driving are to be established using appropriate methods. During driving, the sum of the stresses in the pile due to the impact of the hammer (the dynamic stresses) and the stresses due to axial load and bending (static stresses) is not to exceed the minimum yield stress of the steel. Further, the dynamic stresses are not to exceed 80 to 90 percent of the minimum yield stress of the steel.

If piles are installed by suction, the suction pressure required to reach the target penetration is to provide a minimum factor of 1.5 against soil heaving inside the pile. The pile capacity to resist hydrostatic collapse during installation under the differential pressure is to be evaluated using the procedures given in the *ABS Guide for Buckling and Ultimate Strength Assessment for Offshore Structures*.

11 Gravity Structures

11.1 General

For foundation systems consisting of gravity structures, the stability of the foundation with regard to the bearing and sliding failure modes is to be investigated using the soil shear strengths determined in accordance with 5-4/5.9 and 5-4/7.3. The effects of adjacent structures and the variation of soil properties in the horizontal direction are to be considered where relevant.

Where leveling of the site is not carried out, the predicted tilt of the overall foundation is to be based on the average bottom slope of the sea floor and the tolerance of the elevation measuring device used in the site investigation program. Differential settlement is also to be calculated and the tilting of the structure caused by this settlement is to be combined with the predicted structural tilt. Any increased loading effects caused by the tilting of the structure are to be considered in the foundation stability requirements described in 5-4/11.3.

When an under-pressure or over-pressure is experienced by the sea floor under the structure, provision is to be made to prevent piping which could impair the integrity of the foundation. The influence of hydraulic and slope instability, if any, is to be determined.

Initial consolidation and secondary settlements, as well as permanent horizontal displacements, are to be calculated.

11.3 Stability

The bearing capacity and lateral resistance are to be calculated under the most unfavorable combination of loads. Possible long-term redistribution of bearing pressures under the base slab are to be considered in order to verify that the maximum edge pressures are used in the design of the perimeter of the base.

The lateral resistance of the foundation is to be investigated with respect to various potential shearing planes. Special consideration is to be given to any layers of soft soil.

Calculations for overturning moment and vertical forces induced by the passage of a wave are to include the vertical pressure distribution across the top of the foundation and along the sea floor.

The capacity of the foundation to resist a deep-seated bearing failure is to be analyzed. In lieu of a more rigorous analysis, the capacity of the foundation to resist a deep-seated bearing failure can be calculated by standard bearing capacity formulas applicable to eccentrically loaded shallow foundations, provided that

- i) Uniform soil conditions are present or conservatively chosen soil properties are used to approximate a non-uniform soil condition;
- ii) A trapezoidal distribution of soil pressure is a reasonable expectation,. Alternatively, slip-surface methods, covering a range of kinematically possible deep rupture surfaces can be employed in the bearing capacity calculations.

The maximum allowable shear strength of the soil is to be determined by dividing the ultimate shear strength of the soil by a factor of safety. When the ultimate soil strength is determined by an effective stress method, the safety factor is to be applied to both the cohesive and frictional terms. If a total stress method is used, the safety factor is to be applied to the undrained shear strength. The minimum safety factors to be applied when using a standard bearing capacity formulation and various trial sliding failure planes are specified in 5-4/11.3 TABLE 2. The safety factors to be applied when considering the earthquake conditions will be specially considered.

TABLE 2
Factor of Safety for Allowable Shear Strength of Soil

<i>Loading Condition</i>	<i>Factor of Safety</i>
Normal	2.0
Abnormal	1.5

Where present, additional effects of penetrating walls or skirts which transfer vertical and lateral loads to the soil are to be investigated for their contribution to the bearing capacity and lateral resistance.

11.5 Soil Reaction on the Base

For conditions during and after installation, the reaction of the soil against all structural members seated on or penetrating into the sea floor is to be accounted for in the design of these members. The distribution of soil reactions is to be based on the results obtained according to 5-4/5.7. Calculations of soil reactions are to account for any deviation from a plane surface, the load-deflection characteristics of the soil, and the geometry of the base of the structure.

Where applicable, effects of local soil stiffening, nonhomogeneous soil properties, as well as the presence of boulders and other obstructions, are to be accounted for. During installation, consideration is to be given to the possibility of local contact pressures due to irregular contact between the base and the sea floor; these pressures are additive to the hydrostatic pressure.

An analysis of the penetration resistance of structural elements projecting into the sea floor below the foundation structure is to be performed. The design of the ballasting system is to reflect uncertainties associated with achieving the required penetration of the structure. Since the achievement of the required penetration of the platform and its skirts is of critical importance, the highest expected values of soil strength are to be used in the calculation of penetration.

CHAPTER 5 Structure and Foundation Design

SECTION 5 Marine Operations

1 General (1 July 2020)

In this Guide, marine operations generally include the following activities as appropriate to the planned Bottom-founded Offshore Wind Turbine.

- i)* Load-out and transport
- ii)* Pile installation
- iii)* Final field erection and leveling
- iv)* Installation of the Rotor-Nacelle Assembly (RNA)

The unfavorable effects that may be induced in the Support Structure during marine operations are to be accounted for. This Section is intended to address the requirements on considering the influence that marine operations may have on the safety and integrity of the Bottom-founded Offshore Wind Turbine .

3 Documentation (1 July 2020)

The extent of documentation and analysis of marine operations is to be adequate for the size and type of the Support Structure involved, the particular operation being considered, the extent of past experience with similar operations, and the severity of the expected environmental conditions.

A report on the marine operations planned to install the Support Structure is to be developed and submitted for use in association with the review of the analyses required in 5-5/5. The purpose of this report is to demonstrate that the strength and integrity of the Support Structure are not reduced or otherwise jeopardized by the marine operations.

Generally, this report is to contain the following information.

- i)* Description of the marine operations to be performed and the procedures to be employed.
- ii)* Pre-installation verification procedures for the sea-bed condition and contingency procedures
- iii)* For operations which do not govern the design of the Support Structure, a description of the engineering logic, experience, or preliminary calculations supporting this conclusion.
- iv)* For operations which govern design of the offshore wind turbine Support Structure, the assumptions, calculations, and results of the analyses required in 5-5/5.

5 Analysis

5.1 Loads (1 July 2020)

Analyses are to be performed to determine the type and magnitude of the loads and load combinations to which the Support Structure is exposed during the performance of marine operations.

The design conditions (DLC 8.1 in 4-2/3 TABLE 1) for marine operations are to be defined as appropriate by the Fabricator and/or Owner. Particular attention is to be given to inertial, impact, and local loads that are likely to occur during marine operations. In addition, **DLC 8.2** and DLC 8.3 in 4-2/3 TABLE 1 are also to be considered in accordance with Section 7.4.9 of IEC 61400-3-1 (2019).

Where significant fatigue damage occurs during marine operations, it is to be included in calculating the total fatigue life.

5.3 Stability

Analyses are to be performed to verify that the Support Structure, or its means of support where such exist, has sufficient hydrostatic stability and reserve buoyancy to allow for successful execution of all phases of marine operations.

For the large or unusual Support Structure, an experimental determination of the center of gravity of the structure and its means of support, where such exist, is to be performed.

CHAPTER 6 Equipment and Systems

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CHAPTER 6 Equipment and Systems

SECTION 1 Design Requirements (1 July 2020)

1 General

The requirements for the onboard equipment and systems for the Bottom-founded Offshore Wind Turbine are given in the Section.

In addition, equipment and systems used for Self-Elevating Units are to comply with the *MOU Rules*, where applicable.

3 Fire Fighting Systems and Equipment

Fire fighting systems and equipment are to be designed with consideration of the size, type, and intended service of the Bottom-founded Offshore Wind Turbine. In all cases, the selection of the fire extinguishing medium is to be based on the fire hazard for the space protected. The fire extinguishers are to be visible and readily accessible. Attention is drawn to the relevant requirements of the coastal State or other governmental authorities having jurisdiction over the Bottom-founded Offshore Wind Turbine to be installed.

5 Safety Outfitting

Safety outfitting and measures are to be designed with suitable consideration of the nature of unmanned installations. It is suggested that the coastal State or other governmental authorities having jurisdiction over the Bottom-founded Offshore Wind Turbine are to be installed be contacted to obtain applicable requirements.

7 Identification Marks

A unique name or number is to be assigned to each Bottom-founded Offshore Wind Turbine, which is to conform to requirements of the coastal State or other governmental authorities having jurisdiction over the Bottom-founded Offshore Wind Turbine. This name or number is to be permanently displayed on the structure and will be entered in the *ABS Record*.

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CHAPTER 7 Surveys

SECTION 1 Surveys During Construction, Installation, and Commissioning

1 Overview

1.1 Scope (1 July 2020)

This Section pertains to surveys during the construction and installation of the Bottom-founded Offshore Wind Turbine. The requirements of 7-1/1 are to apply to all Bottom-founded Offshore Wind Turbines covered by this Guide regardless of type.

Additional specific requirements are contained in 7-1/3 for steel **substructures** and in 7-1/5 for concrete **substructures**.

The phases of construction covered by this Section include material manufacture, fabrication, installation and final field erection of the foundation, substructure, tower, and RNA.

For the RNA and tower, the survey scope includes the following two situations:

- The optional **RNA** notation is requested by the Owner. The RNA and tower including its end connections are type approved by ABS and are within the scope of classification and survey.
- The optional **RNA** notation is not requested by the Owner. However, the tower and its connection to the RNA and/or to the substructure are not included in the wind turbine type certificate. In this scenario, the tower and its connection are within the scope of classification and survey. The RNA with a wind turbine type certificate is not within the scope of classification.

1.3 Quality Control Program (1 July 2020)

A quality control program compatible with the type and size of the planned Bottom-founded Offshore Wind Turbine is to be developed and submitted to ABS for review. ABS will review, approve and, as necessary, request modification of this program. The Fabricator is to work with the attending Surveyor to establish the required hold points on the quality control program to form the basis for all future surveys at the fabrication yard. As a minimum, the items enumerated in the various applicable Subsections below are to be covered by the quality control program. Surveyors will be assigned to monitor the fabrication of items within the scope of classification, and to verify that competent personnel are carrying out all tests and inspections specified in the quality control program. It is to be noted that the monitoring provided by ABS is a supplement to and not a replacement for inspections to be carried out by the Fabricator or Owner.

1.5 Access and Notification

During construction, the Surveyor is to have access to structures at all reasonable times. The attending Surveyor is to be notified as to when and where parts of the structure may be examined. If, at any visit, the Surveyor finds occasion to recommend repairs or further inspection, notice is to be made to the Fabricator or its representatives.

1.7 Identification of Materials (1 July 2020)

The Fabricator is to maintain a system of material traceability to the satisfaction of the attending Surveyor, for all Special and Primary Application Structures. Data as to place of origin and results of relevant material tests for structural materials are to be retained and made readily available during all stages of construction (see 7-1/3.23 and 7-1/5.15). Such data are to be available to the Surveyor upon request.

3 Steel Structures

3.1 Quality Control Program (1 July 2020)

The quality control program (see 7-1/1.3) for the construction of the steel Support Structure is to include the following items, as appropriate.

- i) Material quality and traceability
- ii) Steel Forming
- iii) Welder qualification and records
- iv) Welding procedure specifications and qualifications
- v) Weld inspection
- vi) Tolerances alignments and compartment testing
- vii) Corrosion control systems
- viii) Tightness and hydrostatic testing procedures
- ix) Nondestructive testing
- x) Installation of the Support Structure

The items which are to be considered for each of the topics mentioned above are indicated in 7-1/3.

3.3 Material Quality and Traceability

The properties of the material are to be in accordance with Chapter 2, Section 1 of this Guide. Manufacturer's certificates are to be supplied with the material. Verification of the material's quality is to be done by the Surveyor at the plant of manufacture, in accordance with Section 2-1-1 of the *ABS Rules for Materials and Welding (Part 2)*. Alternatively, material manufactured to recognized standards may be accepted in lieu of the above Steel Requirements provided the substitution of such materials is approved by ABS. Materials used are to be in accordance with those specified in the approved design. All materials required for classification purposes are to be tested in the presence of the ABS Surveyor. The Fabricator is to maintain a material traceability system for all the Primary and Special Application Structures.

3.5 Steel Forming

When forming changes base plate properties beyond acceptable limits, appropriate heat treatments are to be carried out to re-establish required properties. Unless approved otherwise, the acceptable limits of the re-established properties are to meet the minimums specified for the original material before forming. As required, ABS will survey formed members for their compliance with the forming dimensional tolerances required by the design.

3.7 Welder Qualification and Records (1 July 2020)

Welders who will work on the Bottom-founded Offshore Wind Turbine are to be qualified in accordance with the welder qualification tests specified in a recognized code or, as applicable, the *ABS Rules for Materials and Welding (Part 2)* to the satisfaction of the attending Surveyor. Certificates of qualification are to be prepared to record evidence of the qualification of each welder qualified by an approved standard/code, and such certificates are to be made available to the Surveyor. If welders have been previously tested in accordance with the requirements of a recognized code and provided that the period of effectiveness of the previous testing has not lapsed, these **previous** welder qualification tests may be accepted.

3.9 Welding Procedure Specifications and Qualifications

Welding procedures are to be approved in accordance with the *ABS Rules for Materials and Welding (Part 2)*. Welding procedures conforming to the provisions of a recognized code may, at the Surveyor's discretion, be accepted. A written description of all procedures previously qualified may be employed in the structure's construction provided it is included in the quality control program and made available to the Surveyor. When it is necessary to qualify a welding procedure, this is to be accomplished by employing the methods specified in the recognized code, and in the presence of the Surveyor.

3.11 Weld Inspection

As part of the overall quality control program, a detailed plan for the inspection and testing of welds is to be prepared. This plan is to include the applicable provisions of this Section.

3.13 Tolerances and Alignments

The overall structural tolerances, forming tolerances, and local alignment tolerances are to be commensurate with those considered in developing the structural design. Inspections are to be carried out to verify that the dimensional tolerance criteria are being met. Particular attention is to be paid to the out-of-roundness of members for which buckling is an anticipated mode of failure. Structural alignment and fit-up prior to welding are to be monitored to verify consistent production of quality welds.

3.15 Corrosion Control Systems

The details of any corrosion control systems employed for the structure are to be submitted for review. Installation and testing of the corrosion control systems are to be carried out to the satisfaction of the attending Surveyor in accordance with the approved plans.

3.17 Tightness and Hydrostatic Testing Procedures

Compartments, which are designed to be permanently watertight or to be maintained watertight during installation, are to be tested by a procedure approved by the attending Surveyor. The testing is also to be witnessed by the attending Surveyor.

3.19 Nondestructive Testing

A system of nondestructive testing is to be included in the fabrication specification of the structures. The minimum extent of nondestructive testing is to be in accordance with this Guide or recognized design Code. All nondestructive testing records are to be reviewed and approved by the attending Surveyor. Additional nondestructive testing may be requested by the attending Surveyor if the quality of fabrication is not in accordance with industry standards.

3.21 Testing and Trials of a Self-Elevating Unit to be Used as the Substructure (1 July 2020)

A Self-Elevating Mobile Offshore Unit intended to be converted to the site dependent **substructures** of an offshore wind turbine is to be subject to the applicable surveys during construction and installation specified in this Guide in addition to the testing and trials required by **Part 7, Chapter 1** of the *ABS Rules for Building and Classing Mobile Offshore Units*.

3.23 Records

A data book of the records of construction activities is to be developed and maintained so as to compile a record as complete as practicable. The pertinent records are to be adequately prepared and indexed to assure their usefulness, and they are to be stored so that they may be easily recovered.

For a steel structure, the construction record is to include, as applicable, the following:

- i) Material traceability records including mill certificates
- ii) Welding procedure specification and qualification records
- iii) Shop welding practices
- iv) Welding inspection records
- v) Construction specifications
- vi) Structural dimension check records
- vii) Nondestructive testing records
- viii) Records of completion of items identified in the quality control program
- ix) Towing and pile driving records
- x) Position and orientation records
- xi) Leveling and elevation records, etc.

The compilation of these records is a condition of classification.

After fabrication and installation, these records are to be retained by the Owner or Fabricator for future references. The minimum time for record retention is not to be less than the greatest of the following: the warranty period, the time specified in construction agreements, or the time required by statute or governmental regulations.

5 Concrete Structures

5.1 Quality Control Program (1 July 2020)

The quality control program (see 7-1/1.3) for a concrete structure is to cover the following items, as appropriate.

- i) Inspections prior to concreting
- ii) Inspection of batching, mixing and placing concrete
- iii) Inspections of form removal and concrete curing
- iv) Inspection of prestressing and grouting
- v) Inspection of joints
- vi) Inspection of finished concrete
- vii) Installation of the Support Structure
- viii) Tightness and hydrostatic testing as applicable (see 7-1/3.17)

The items which are to be considered for each of the topics mentioned above, except for viii), are indicated in 7-1/5.3 through 7-1/5.13.

5.3 Inspections Prior to Concreting

Prior to their use in construction, the manufacturers of cement, reinforcing rods, prestressing tendons and appliances are to provide documentation of the pertinent physical properties. These data are to be made available to the attending Surveyor for verification of conformity with the properties specified in the approved design.

As applicable, at the construction site, the Surveyor is to be satisfied that proper consideration is being given to the support of the structure during construction, the storage of cement and prestressing tendons in weathertight areas, the storage of admixtures and epoxies to manufacturer's specifications, and the storage of aggregates to limit segregation, contamination by deleterious substances and moisture variations within the stock pile.

Forms and shores supporting the forms are to be inspected to verify that they are adequate in number and type, and that they are located in accordance with the approved plans. The dimensions and alignment of the forms are to be verified by the attending Surveyor. The measurements are to be within the allowable finished dimensional tolerances specified in the approved design.

Reinforcing steel, prestressing tendons, post-tensioning ducts, anchorages and any included steel are to be checked, as appropriate to the planned structure, for size, bending, spacing, location, firmness of installation, surface condition, vent locations, proper duct coupling, and duct capping.

5.5 Inspection of Batching, Mixing, and Placing Concrete

The production and placing of the concrete are to employ procedures which will provide a well-mixed and well-compacted concrete. Such procedures are also to limit segregation, loss of material, contamination, and premature initial set during all operations.

Mix components of each batch of concrete are to be measured by a method specified in the quality control program. The designer is to specify the allowable variation of mix component proportions, and the Fabricator is to record the actual proportions of each batch.

Testing during the production of concrete is to be carried out following the procedures specified in the quality control program. As a minimum, the following concrete qualities are to be measured by the Fabricator.

- i) Consistency
- ii) Air content
- iii) Density or Specific Gravity
- iv) Strength

Field testing of aggregate gradation, cleanliness, moisture content, and unit weight is to be performed by the Fabricator following standards and schedules specified in the quality control program. The frequency of testing is to be determined with consideration of the uniformity of the supply source, volume of concreting, and variations of atmospheric conditions. Mix water is to be tested for purity following the methods and schedules specified in the quality control program.

5.7 Inspections of Form Removal and Concrete Curing

The structure is to have sufficient strength to bear its own weight, construction loads and the anticipated environmental loads without undue deformations before forms and form supports are removed. The schedule of form removal is to be specified in the quality control program, giving due account to the loads and the anticipated strength.

Curing procedures for use on the structure are to be specified in the quality control program. When conditions at the construction site cause a deviation from these procedures, justification for these deviations is to be fully documented and included in the construction records.

Where the construction procedures require the submergence of recently placed concrete, special methods for protecting the concrete from the effects of salt water are to be specified in the quality control program. Generally, concrete is not to be submerged until 28 days after placing (see also 5-3/11.3.5).

5.9 Inspection of Prestressing and Grouting

A schedule indicating the sequence and anticipated elongation and stress accompanying the tensioning of tendons is to be prepared. Any failures to achieve proper tensioning are to be immediately reported to the designer to obtain guidance on needed remedial actions.

Pre- or post-tensioning loads are to be determined by measuring both tendon elongation and tendon stress. These measurements are to be compared. In the case that the variation of measurements exceed the specified amount, the cause of the variation is to be determined and any necessary corrective actions are to be accomplished.

The grout mix is to conform to that specified in the design. The Fabricator is to keep records of the mix proportions and ambient conditions during grout mixing. Tests for grout viscosity, expansion and bleeding, compressive strength, and setting time are to be made by the Fabricator using methods and schedules specified in the quality control program. Employed procedures are to verify that ducts are completely filled.

Anchorage are to be inspected to verify that they are located and sized as specified in the design. Anchorages are also to be inspected to verify that they will be provided with adequate cover to mitigate the effects of corrosion.

5.11 Inspection of Joints (1 July 2020)

Where required, leak testing of construction joints is to be carried out using procedures specified in the quality control program. When deciding which joints are to be inspected, consideration is to be given to the hydrostatic head on the subject joint during normal operation, the consequence of a leak at the subject joint, and the ease of repair once the Bottom-founded Offshore Wind Turbine is in service.

5.13 Inspection of Finished Concrete

The surface of the hardened concrete is to be comprehensively inspected for cracks, honeycombing, pop-outs, spalling and other surface imperfections. When such defects are found, their extent is to be reported to the Surveyor and to the designer for guidance on any necessary repairs.

The structure is to be examined using a calibrated rebound hammer or a similar nondestructive testing device. Where the results of surface inspection, cylinder strength tests or nondestructive testing do not meet the design criteria, the designer is to be consulted regarding remedial actions to be taken.

The completed sections of the structure are to be checked for compliance to specified design tolerances for thickness, alignment, etc., and to the extent practicable, the location of reinforcing and prestressing steel and post-tensioning ducts. Variations from the tolerance limits are to be reported to the designer for evaluation and guidance on any necessary remedial actions.

5.15 Records (1 July 2020)

Refer to 7-1/3.23 for information on compiling construction records. For the concrete Support Structure of an offshore wind turbine, the construction records are to include, as applicable, the following:

- i) All material certificates and test reports
- ii) Tensioning and grouting records
- iii) Concrete records including weight
- iv) Moisture content and mix proportions
- v) A list of test methods and results

- vi) Ambient conditions during the pours
- vii) Calibration data for test equipment
- viii) Towing records
- ix) Data on initial structural settlements
- x) Inspector's logs

These records are to be retained by the Owner.

7 Installation and Commissioning Surveys (1 July 2020)

7.1 Installation of Steel Substructure and Foundation (1 July 2020)

Installation procedures are to be submitted to ABS for review and approval as described in Section 5-5. The Surveyor is to witness the following activities, as applicable to the planned structure, to ascertain whether they have been accomplished in a manner conforming to the approved procedures.

- i) Installation of the Substructure and Foundation
- ii) Piling and grouting
- iii) Welding and nondestructive testing
- iv) Final field erection and leveling
- v) Pre-tensioning

Significant deviations from approved plans and procedures or any incidents such as excessive tilting of the foundation or abnormal vibrations during pile driving may require re-submittal of supporting documentation to provide an assessment of the significance of deviation and any necessary remedial actions to be taken.

To verify that overstressing of the structure during transportation has not occurred, ABS is to be provided access to towing records to ascertain if conditions during the towing operations exceeded those employed in the design. Results are to be submitted to demonstrate compliance with the reviewed design analysis.

7.3 Installation of Concrete Substructure and Foundation (1 July 2020)

Installation procedures are to be submitted to ABS for review and approval as described in Section 5-5. The Surveyor is to witness the following operations, as applicable to the planned structure, to verify that they have been accomplished in a manner conforming to plans or drawings covering these operations.

- i) Installation
- ii) Final field erection and leveling
- iii) Pre-tensioning

Significant deviations from approved plans and procedures may require re-submittal of supporting documentation to provide an assessment of the significance of the deviation and the remedial actions to be taken.

To verify that overstressing of the structure during transportation has not occurred, ABS is to have access to towing records to ascertain if conditions during the towing operations exceeded those employed in the design. Results are to be submitted to demonstrate compliance with the reviewed design analysis.

7.5 Installation of Rotor-Nacelle Assemblies (RNAs) and Tower (1 July 2020)

Where the RNA and tower are not within the scope of classification, the valid type certificate of the RNA and tower is to be examined by the attending Surveyors. The installation of the RNA and tower and the

hook-up of the power cable system is not to damage the interface structure to the Substructure and the Foundation.

Where the RNA and tower are within the scope of classification, the Surveyor is to witness the installation of at least one (1) RNA and one (1) tower per each wind turbine type. Where there are more than fifty (50) RNAs and towers of the same type, at least one (1) RNA and one (1) tower installation per every fifty (50) wind turbines of the same type are to be witnessed by the Surveyor. The selection of the RNA and tower installations to be witnessed is to reflect having Surveys at the start and end of RNA and tower installation periods and the rate of installation within an installation period.

The attending Surveyor is to verify that the RNA and tower to be installed are in compliance with the relevant design documents for the Bottom-founded Offshore Wind Turbine. Deviations from approved design documents and plans or any incidents such as damage or overstress to the Bottom-founded Offshore Wind Turbine during the installation may require re-submittal of supporting documentation to provide an assessment of the significance of deviation and any necessary remedial actions to be taken.

7.7 Commissioning Surveys (1 July 2020)

The commissioning date will be the date on which the Surveyor issues the Interim Classification Certificate for the Bottom-founded Offshore Wind Turbine. Commissioning of all ABS Rule/Guide-required systems is to be verified by the attending ABS Surveyor. The commissioning is to be in accordance with the approved step-by-step commissioning procedures. The Surveyor is to be permitted access to critical/hold points to verify that the procedures are satisfactorily accomplished. The Surveyor is to observe the Bottom-founded Offshore Wind Turbine operating under various capacities and conditions.

Where the RNAs are within the scope of classification, the Surveyor is to witness the commissioning of at least one (1) RNA per each wind turbine type. Where there are more than fifty (50) RNAs of the same type, the commissioning of at least one (1) additional RNA per every fifty (50) turbines of the same type is to be witnessed by the Surveyor. The selection of RNA commissioning to be witnessed is to reflect having Surveys at the start and end of RNA commissioning periods. Approved turbine RNA operations including emergency procedures are to be verified to the extent deemed necessary by the attending Surveyor. Overall performances of the turbine RNA are to be verified for compliance with the design parameters used in the design of the Bottom-founded Offshore Wind Turbine. Records of all these performances are to be maintained and made available to ABS.

7.9 Personnel Safety (1 July 2020)

Verification of personnel safety features which include checks of operational readiness of all lifesaving, fire detection and firefighting equipment, emergency shutdown systems, unobstructed escape routes and establishment of communication procedures are to be taken during commissioning and are required to be verified by the attending Surveyor.

CHAPTER 7 Surveys

SECTION 2 Surveys After Construction

1 Condition for Surveys after Construction (1 July 2020)

1.1 Damage, Failure, and Repair

1.1.1 Examination and Repair (1 July 2020)

Damage, failure, deterioration or repair to a classed Bottom-founded Offshore Wind Turbine, which affects or may affect classification, is to be submitted by the Owner or their representatives for examination by the Surveyor at the first opportunity. All repairs found necessary by the Surveyor are to be carried out to the Surveyor's satisfaction.

1.1.2 Repairs on Site

Where repairs to the structure, which affect or may affect classification, are intended to be carried out at site, complete repair procedure including the extent of proposed repair and the need for Surveyor's attendance on site is to be submitted to and agreed upon by the Surveyor reasonably in advance. The above is not intended for routine maintenance.

1.1.3 Representation

Nothing contained in this Section or in a regulation of any government or other administration, or the issuance of any report or certificate pursuant to this Section or such a rule or regulation, is to be deemed to enlarge upon the representations expressed in 1-1-1/1 through 1-1-1/7 of the ABS *Rules for Conditions of Classification - Offshore Units and Structures (Part 1)*.

1.3 Notification and Availability for Survey (1 July 2020)

The Surveyor is to **be provided** access to a classed Bottom-founded Offshore Wind Turbine at all reasonable times. The Owners or their representatives are to notify the Surveyor on all occasions when the Bottom-founded Offshore Wind Turbine can be examined on site.

The Surveyor is to undertake all surveys on the classed Bottom-founded Offshore Wind Turbine upon request, with adequate notification, from the Owners or their representatives and is to report thereon to ABS. Should the Surveyor find occasion during any survey to recommend repairs or further examination, notification is to be given immediately to the Owners or their representatives in order that appropriate actions may be taken. The Surveyor is to avail himself of every convenient opportunity for performing periodical surveys in conjunction with surveys of damages and repairs in order to avoid duplication of work. Also see 1-1-8/3 of the ABS *Rules for Conditions of Classification – Offshore Units and Structures (Part 1)*.

1.5 In-Service Inspection Plan (1 July 2020)

The surveys are to be carried out in accordance with the reviewed In-Service Inspection Plan to confirm that the Bottom-founded Offshore Wind Turbine remains in compliance with the applicable classification requirements and other relevant standards. The In-Service Inspection Plan as required by 1-1/9.5 is to cover all surveys for the design life of the Bottom-founded Offshore Wind Turbine.

As an alternative, a Risk Based Inspection (RBI) plan approved in accordance with 7-2/19 of this Guide may be considered so long as it includes all components of the regular inspection program. The RBI plan is to be submitted and agreed by ABS.

1.7 Annual Surveys (1 July 2020)

Annual Surveys of the Bottom-founded Offshore Wind Turbine are to be made within three months either way of each annual anniversary date of crediting of the previous Special Periodical Survey or original construction date. Where the Surveyor is engaged in the survey of a grouping of structures of similar design and location, and where requested by the Owner, special consideration will be given to the timing of Annual Surveys and Special Periodical Surveys such that all periodical survey due dates can be harmonized.

For the Annual Survey of a wind farm with a group of wind turbines having a similar design, the group of wind turbines can be considered as a set and examined in accordance with the approved In-Service Inspection Plan.

1.9 Special Periodical Surveys (1 July 2020)

Special Periodical Surveys of the Bottom-founded Offshore Wind Turbine are to be carried out at least once every five years. If a Special Periodical Survey is not completed at one time, it will be credited as of the completion date of the survey provided the due date of the Special Periodical Survey is not overdue by more than six months. Where the Surveyor is engaged in the survey of a grouping of structures of similar design and location, and where requested by the Owner, special consideration will be given to the timing of Special Periodical Surveys so that the due dates for all periodical surveys can be harmonized.

For Special Periodical Surveys of an offshore wind farm with a group of wind turbines having a similar design, the group of wind turbines can be surveyed as a set, in accordance with the In-Service Inspection Plan. Continuous Surveys can be applied (see 7-2/7).

1.11 Continuous Surveys

At the request of the Owner, and upon approval of the proposed arrangement, a system of Continuous Surveys may be undertaken whereby all the Special Periodical Survey requirements are carried out in regular rotation and completed within the normal Special Periodical Survey interval.

For Continuous Surveys, a suitable notation will be entered in the ABS *Record* and the date of completion of the cycle published. If any defects are found during these surveys, they are to be examined and dealt with to the satisfaction of the Surveyor.

1.13 Reactivation Surveys (1 July 2020)

When the Bottom-founded Offshore Wind Turbine has been out of service for an extended period, the requirements for surveys on reactivation are to be specially considered in each case. Due regard is to be given to the status of surveys at the time of the commencement of the deactivation period, the length of the period, and conditions under which the Bottom-founded Offshore Wind Turbine had been maintained during that period.

1.15 Incomplete Surveys

When a survey is not completed, the Surveyor is to report immediately upon the work done in order that the Owner and ABS may be advised of the remaining parts to be surveyed.

1.17 Alterations (1 July 2020)

No major alterations, which affect classification of the installation, are to be made to a classed Bottom-founded Offshore Wind Turbine unless plans of the proposed alterations are submitted and approved by ABS before the alterations are undertaken. Such alterations are to be carried out to the satisfaction of the Surveyor. Nothing contained in this Section or in a rule or regulation of any government or other administration, or the issuance of any report or certificate pursuant to this Section or such a rule or regulation, is to be deemed to enlarge upon the representations expressed in 1-1-1/1 through 1-1-1/7 of the *ABS Rules for Conditions of Classification - Offshore Units and Structures (Part 1)* and the issuance and use of any such reports or certificates are to be, in all respects, governed by 1-1-1/1 through 1-1-1/7 of the above Part 1.

1.19 Survey of a Self-Elevating Unit to be Used as the Substructure (1 July 2020)

A Self-Elevating Mobile Offshore Unit that has been converted to the site-dependent **substructure** of an offshore wind turbine is to be subject to the surveys after construction required in this Guide in addition to the applicable surveys of structures required by **Part 7, Chapter 2** of the *ABS Rules for Building and Classing Mobile Offshore Units*.

1.21 Survey for Extension of Use (1 July 2020)

Existing installations to be used at the same location for an extended period of time beyond their original design life are subject to additional surveys, as deemed necessary, to determine the actual condition of the Bottom-founded Offshore Wind Turbine. The extent of the survey will depend on the completeness of the existing survey documents. ABS will review and verify maintenance manual, logs and records. Any alterations, repairs or installation of equipment since installation are to be included in the records.

Those survey requirements in **7-2/1.9** for the Special Periodical Survey have to be included in the survey for extension of use. The surveys generally cover examination of splash zone, inspection of above water and underwater structural members and welds for damages and deteriorations, examination and measurements of corrosion protection systems and marine growth, sea floor condition survey, examination of secondary structural attachments such as J-Tube, service decks, etc. Special attention is to be given to the following critical areas.

- i) Areas of high stress
- ii) Areas of low fatigue life
- iii) Damage incurred during installation or while in service
- iv) Repairs or modifications made while in service
- v) Abnormalities found during previous surveys

An inspection report of the findings is to be submitted to ABS for review and evaluation of the condition of the Bottom-founded Offshore Wind Turbine.

The need for more frequent future periodical surveys is to be determined based on the calculated remaining fatigue life described in Section 4-1-2 of the *ABS Rules for Building and Classing Offshore Installations*, with the exception that the load, strength and fatigue calculations are to follow Chapters 4 and 5 of this Guide as well as past inspection results.

3 Annual Surveys (1 July 2020)

Each Annual Survey is to include a thorough visual examination of above water structure. Special attention will be given to the splash zone for possible damage or deterioration from corrosion. Additionally, where it appears that significant deterioration or damage has occurred to an installation since the last survey, a general examination, by diver, underwater camera, submersible, or other suitable means, of the underwater structure, the scour protection, the sea floor, and the corrosion control system is to be carried out. Underwater examinations are to be contracted by the Owner and monitored by the Surveyor.

Any novel features incorporated in the design are to be given special attention according to procedures agreed to during review of the design.

Particular attention is to be given to significant modifications or repairs made as a result of findings at the previous survey.

The Annual Survey is also to include verification that the approved design life has not been exceeded. The Surveyor is to confirm the design life limits. The ABS technical office is to be consulted for verification. If the end of the design life has been reached, the provisions of Section 4-1-2 of the *ABS Rules for Building and Classing Offshore Installations*, with the exception that the load, strength and fatigue calculations are to follow Chapters 4 and 5 of this Guide, are to be applied and specific requirements for maintaining the class of the Bottom-founded Offshore Wind Turbine are to be obtained from ABS.

5 Special Periodical Surveys (1 July 2020)

The requirements of the Annual Survey are to be met during the Special Periodical Survey. Additionally, underwater inspection of selected areas is to be carried out. In addition, nondestructive testing is to be carried out on representative joints of the structures and if found necessary, structural supports of J-Tubes. The extent and methods to be employed in such testing, cleaning, and inspection of the structure are to be in accordance with an approved inspection plan. The inspection plan, which is to be submitted for approval, is to cover all Special Periodical Surveys for the design life of the Bottom-founded Offshore Wind Turbine. It is to enumerate in detail the items to be surveyed, the testing and inspection procedures to be employed, and where necessary, cleaning and nondestructive testing procedures. The plan is to include sufficiently detailed drawings which can be used by the Surveyor to reference and locate the items to be surveyed. The testing, cleaning, and inspection services are to be provided by the Owner and monitored by the Surveyor. Divers carrying out structural inspections and nondestructive testing on the structures are to be suitably qualified.

In addition, the Special Periodical Survey is to include monitoring of the effectiveness of the corrosion protection system. The effectiveness of the corrosion protection system is to be monitored by taking measurements of the potential voltages generated by such systems. Scour in way of foundation legs and tilt of the Bottom-founded Offshore Wind Turbine are also to be checked and witnessed by the attending Surveyor.

The Special Periodical Survey is also to include verification that the approved design life has not been exceeded. The Surveyor is to confirm the design life limits and the ABS technical office is to be consulted for verification. If the end of the design life has been reached, the provisions of Section 4-1-2 of the *ABS Rules for Building and Classing Offshore Installations*, with the exception that the load, strength and fatigue calculations are to follow Chapters 4 and 5 of this Guide, are to be applied and specific requirements for maintaining the class of Bottom-founded Offshore Wind Turbine are to be obtained from ABS.

7 Continuous Surveys (1 July 2020)

As an alternative arrangement for the Special Periodical Surveys, the Continuous Surveys are to be applied annually to a minimum of 20% of the Bottom-founded Offshore Wind Turbines selected from the offshore wind farm. All the Special Periodical Survey requirements as specified in 7-2/5 are to be fulfilled during the Continuous Surveys. Each Bottom-founded Offshore Wind Turbine in the offshore wind farm is to be subject to at least once Special Periodical Survey within any five consecutive years. Where significant deterioration or damage has occurred to the installations or specific problems are identified as an indication of a serial defect, the Continuous Surveys are to be carried out to the satisfaction of the Surveyor for an increased number of the Bottom-founded Offshore Wind Turbines.

9 Gauging

Thickness gauging is required to be taken at each Special Periodical Survey. Suspect areas including structures in way of the splash zone are to be tested for thickness and results submitted to the attending Surveyor for review.

11 Structural Deterioration (1 July 2020)

Where thickness measurement and visual examination show evidence of significant structural deterioration, the structural integrity of the structure for continuous use is to be justified by engineering analyses. Deteriorated structural members are to be modeled such that they will add hydrodynamic or wind loads to the Bottom-founded Offshore Wind Turbine, but will not contribute to the strength of the Bottom-founded Offshore Wind Turbine. Results of these analyses are to be submitted to ABS for review and approval. If the results show that structural integrity is inadequate, the deteriorated structural components are to be suitably reinforced or replaced with new materials having required dimensions and properties in accordance with approved procedure.

13 Marine Growth (1 July 2020)

During any Annual or Special Periodical Survey, assessment of the degree of marine growth is to be carried out. Marine growth is to be removed where it is found to be thicker than the original approved design. If the Owner decides to leave the marine growth greater than what is allowed in the approved design, the Owner is to justify that the higher hydrodynamic loading due to the additional marine growth will not affect the structural integrity of the structure. The Owner is to submit **as a minimum**, an in-place analysis to justify that the installation is capable of withstanding environmental wave loads resulting from the maximum marine growth that the Owner is prepared to maintain.

15 Maintenance of Rotor-Nacelle Assembly (1 July 2020)

The Owner is to submit an annual report for review by the attending ABS Surveyor attesting to the following:

- i) Maintenance has been carried out by authorized and qualified personnel in accordance with the maintenance manual.
- ii) The control settings have been checked with regard to conformance with the limiting values specified in design documentation of the classed Bottom-founded Offshore Wind Turbine.
- iii) All repair, modification and replacement have been carried out in accordance with the manufacturers' recommendations and have not altered values of RNA parameters specified in design documentation of the classed Bottom-founded Offshore Wind Turbine.

17 Statutory Certification (15 January 2013)

When ABS is authorized to perform surveys on behalf of a coastal State or a governmental authority, and when requested by the Owner, items as specified by the coastal State, the governmental authority or Owner will be surveyed. Reports indicating the results of such surveys will be issued accordingly. Where the periodicity and types of surveys on behalf of a coastal State or a governmental authority differ from those required by the applicable portions of this Section, coastal State or governmental authority's requirements take precedence.

19 Risk-based Surveys for Maintenance of Class (1 July 2020)

A properly developed risk-based inspection plan or reliability centered maintenance plan may be credited as satisfying requirements of surveys for maintenance of Class for the Bottom-founded Offshore Wind Turbine. The plan is to be developed in accordance with the *ABS Guide for Risk-Based Inspection for Floating Offshore Installations* or the *ABS Guide for Surveys Based on Machinery Reliability and Maintenance Techniques*.

The application of the Guides referenced above in this Subsection does not cover any statutory survey requirements that may apply to the Bottom-founded Offshore Wind Turbine being considered. Only when the governmental authorities accept and authorize ABS for certification based on risk-based inspection techniques, ABS will carry out such surveys, as authorized. The Owner is to ensure that in developing the inspection plan, due consideration is given to applicable requirements external to ABS. If the governmental authorities do not accept a risk-based approach, surveys will be carried out in a conventional, prescriptive manner in compliance with 7-2/1 through 7-2/19.

APPENDIX 1

Wind Spectra and Coherence Functions

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APPENDIX 1

Wind Spectra and Coherence Functions

SECTION 1

Wind Spectra and Coherence Functions

1 Kaimal Wind Spectrum and Exponential Coherence Model (1 July 2020)

A modified version of the Kaimal wind spectrum is provided in IEC 61400-1 (2019). The power spectral densities for the wind fluctuations in three dimensions are given as follows:

$$\frac{f \cdot S_k(f)}{\sigma_k^2} = \frac{4fL_k/V_{hub}}{(1 + 6fL_k/V_{hub})^{5/3}}$$

where

$S_k(f)$ = spectral energy density at frequency f , in $\text{m}^2\text{s}^{-2}/\text{Hz}$ ($\text{ft}^2\text{s}^{-2}/\text{Hz}$)

f = frequency, in Hz

k = index referring to the direction of wind speed component (i.e., 1 = longitudinal, 2 = lateral, and 3 = upward, as depicted in 3-2/3.1 FIGURE 1)

σ_k = standard deviation of turbulent wind speed component (see A1-1/1 TABLE 1)

$$= \left[\int_0^{\infty} S_k(f) df \right]^{0.5}$$

L_k = integral parameter of turbulent wind speed component (see A1-1/1 TABLE 1)

TABLE 1
Spectral Parameters for the Kaimal model

	Wind Speed Direction		
	$k=1$ (longitudinal)	$k=2$ (lateral)	$k=3$ (upward)
Standard Deviation (σ_k)	σ_1	$0.8\sigma_1$	$0.5\sigma_1$
Integral Scale (L_k)	$8.1\Lambda_1$	$2.7\Lambda_1$	$0.66\Lambda_1$

Note:

- σ_i = standard deviation of longitudinal turbulent wind speed
 Λ_1 = scale parameter of the turbulence as specified in 3-2/11.5

Along with the Kaimal spectrum, an exponential coherence model is provided in IEC 61400-1 (2019) to account for the spatial correlation of the longitudinal wind speed:

$$Coh(r, f) = \exp \left[-12 \sqrt{(f \cdot r / V_{hub})^2 + (0.12 \cdot r / L_c)^2} \right]$$

where

- $Coh(r, f)$ = coherence function at frequency f
 f = frequency, in Hz
 r = magnitude of the projection of the separation vector between the two points on to a plane normal to the average wind direction, in m (ft)
 V_{hub} = 10-minute mean wind speed at hub height, in m/s (ft/s)
 L_c = coherence scale parameter, in m (ft)
= $8.1\Lambda_1$, where Λ_1 is specified in 3-2/11.3

3 NPD (Frøya) Wind Spectrum and Coherence Model (1 July 2020)

For the extratropical storm wind, the NPD (Frøya) wind spectrum in conjunction with the two-point coherence function recommended by API RP 2MET (2019) may be applied.

3.1 Wind Spectrum

The following wind spectrum is defined for the energy density of the longitudinal wind speed fluctuations:

$$S(f) = \frac{320\phi^2 \left(\frac{U_0}{10\phi}\right)^2 \left(\frac{z}{10\phi}\right)^{0.45}}{(1 + \tilde{f}^n)^{(5/3n)}}$$

$$\tilde{f} = 172f \left(\frac{z}{10\phi}\right)^{2/3} \left(\frac{U_0}{10\phi}\right)^{-0.75}$$

where

- $S(f)$ = spectral energy density at frequency f , in $\text{m}^2\text{s}^{-2}/\text{Hz}$ ($\text{ft}^2\text{s}^{-2}/\text{Hz}$)
 f = frequency, in Hz
 U_0 = 1-hour mean wind speed at 10 m (32.8 ft) above the SWL, in m/s (ft/s)
 n = 0.468
 z = height above the SWL, in m (ft)
 ϕ = unit conversion factor
= 1 when using SI units (m, m/s)
= 3.28 when using US Customary units (ft, ft/s)

3.3 Spatial Coherence

The squared correlation between the spectral energy densities $S(f)$ of the longitudinal wind speed fluctuations between two points (x_j, y_j, z_j) , $j = 1, 2$, in space is described by the two-point coherence function as follows:

$$Coh(f) = \exp\left\{-\frac{1}{U_0/\phi} \cdot \left[\sum_{i=1}^3 A_i^2\right]^{\frac{1}{2}}\right\}$$

$$A_i = \alpha_i f^{r_i} \left(\frac{\Delta_i}{\phi}\right)^{q_i} \left(\frac{\sqrt{z_1 z_2}}{10\phi}\right)^{-p_i}$$

where the coefficients α_i , p_i , q_i , r_i and the distances Δ_i are specified in A1-1/Table 2, and

$Coh(f)$	=	coherence function at frequency f
f	=	frequency, in Hz
U_0	=	1-hour mean wind speed at 10 m (32.8ft) above the SWL, in m/s (ft/s)
(x_j, y_j, z_j)	=	spatial coordinates of two points ($j = 1, 2$) where x_j is in the longitudinal (along wind) direction, y_j is in the lateral (across wind) direction and z_j is the height above the SWL in the upward direction, in m (ft)
ϕ	=	unit conversion factor
	=	1 when using SI units (m, m/s)
	=	3.28 when using US Customary units (ft, ft/s)

TABLE 2
Coefficients and Distances for the
Three-dimensional Coherence Function

i	Δ_i	q_i	p_i	r_i	α_i
1	$ x_2 - x_1 $	1.00	0.4	0.92	2.9
2	$ y_2 - y_1 $	1.00	0.4	0.92	45.0
3	$ z_2 - z_1 $	1.25	0.5	0.85	13.0

5 Tropical Cyclone Wind Spectrum (1 July 2020)

For tropical storm wind, the following wind speed spectrum for the energy density of the longitudinal wind speed fluctuations is recommended by API RP 2MET (2019):

$$S(f, z) = \frac{4I_u^2 U_{w, 1h(z)} L_{u, x(z)}}{\left[1 + 70.8(f L_{u, x(z)} / U_{w, 1h(z)})^2\right]^{5/6}}$$

where

$S(f, z)$	=	spectral energy density at frequency f , and height z , in $\text{m}^2\text{s}^{-2}/\text{Hz}$ ($\text{ft}^2\text{s}^{-2}/\text{Hz}$)
f	=	frequency, in Hz
z	=	height above the SWL, in m (ft)

$U_{w,1h}(z)$ = 1-hour average wind speed at a height z above the MSL, in m/s (ft/s)

$I_u(z)$ = turbulence intensity at the height z above the MSL (see A2-1/5), (dimensionless)

$L_{u,x}(z)$ = integral length scale at the height z above the MSL, in m (ft)

$$L_{u,x}(z) = \frac{50\phi\left(\frac{z}{\phi}\right)^{0.35}}{\left(\frac{z_0}{\phi}\right)^{0.063}}$$

z_0 = surface roughness length for tropical cyclone winds (see A2-1/3), in m (ft)

ϕ = unit conversion factor, (dimensionless)

= 1 when using SI units (m, m/s)

= 3.28 when using US Customary units (ft, ft/s)

APPENDIX 2

Tropical Cyclone Wind Speed Profile, Standard Deviation, Turbulence Intensity and Gust Factor

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APPENDIX 2

Tropical Cyclone Wind Speed Profile, Standard Deviation, Turbulence Intensity and Gust Factor

SECTION 1

Tropical Cyclone Wind Speed Profile, Standard Deviation, Turbulence Intensity and Gust Factor

1 Introduction (1 July 2020)

Extensive dropsonde measurements of major tropical cyclones in the Gulf of Mexico indicate that the tropical cyclone wind speed profile is on average steeper than the exponential wind speed profile specified in IEC 61400-1 for Extreme Wind Speed Model (EWM, see 4-2/11.1 of this Guide). The measured wind profiles are also steeper than those **NPD (Frøya) wind model** originally developed based on the North Sea data.

Tropical cyclone wind data collected on the platforms in the Gulf of Mexico indicate that the turbulence intensity is insensitive to the change of mean wind speed. The average observed value of turbulence intensity is slightly smaller than 0.11 as defined in IEC 61400-1 for the Extreme Wind Speed Model (EWM, see 4-2/11.1 of this Guide) but could differ noticeably from the value calculated **with the NPD (Frøya) wind model**. In addition, the measured gust factors of tropical cyclone winds are found to be less sensitive to the change of mean wind speed in comparison to the calculated gust factors in **the NPD (Frøya) wind model**.

The formulation of the wind speed profile, wind speed standard deviation, turbulence intensity and gust factors presented in this Appendix is found to provide reasonably good representations of the measurements of tropical cyclone wind in the Gulf of Mexico. In absence of site data, the formulation in this Appendix may be used to model tropical cyclone (also known as hurricane or typhoon) wind conditions over the open ocean.

3 Tropical Cyclone Wind Speed Profile

Tropical cyclone wind boundary layers within approximately 200 m above the sea surface of the open ocean can be represented by an equilibrium form of the logarithmic boundary layer profile under neutral stability conditions, i.e.:

$$V_{1hr}(z) = \frac{u_*}{k} \ln\left(\frac{z}{z_0}\right)$$

where

$V_{1hr}(z)$	=	one hour mean longitudinal wind speed at the height z above the sea surface, in m/s (ft/s)
u_*	=	friction velocity, in m/s (ft/s)
z	=	height above the sea surface, in m (ft)
z_0	=	surface roughness length, in m (ft)
k	=	von Karman constant, (dimensionless)
	=	0.4

For mean wind speed $V_{1hr}(z_{ref})$ lower than approximately 27 m/s (52.5 knots), the surface roughness length z_0 over the open ocean may be calculated using the Charnock equation:

$$z_0 = \frac{\alpha u_*^2}{g} = \frac{\alpha}{g} \left[\frac{k V_{1hr}(z)}{\ln\left(\frac{z}{z_0}\right)} \right]^2$$

where

α	=	0.011, as suggested in IEC 61400-3, for the open ocean
z_{ref}	=	10 m (33 ft) reference height above the sea surface, in m (ft)
$V_{1hr}(z_{ref})$	=	1-hour mean wind speed at the reference height z_{ref} above the sea surface, in m/s (ft/s)
g	=	acceleration of gravity, in m/s ² (ft/s ²)

Alternatively, the surface roughness length z_0 over the open ocean can be determined from the boundary layer wind profile and expressed in terms of the surface drag coefficient $C_d(z_{ref})$ at the reference height z_{ref} = 10 m (33 ft) through the surface shear stress $\tau_0 = \rho u_*^2 = \rho C_d(z_{ref}) V_{1hr}^2(z_{ref})$, where ρ is the air density and $C_d(z_{ref})$ is the surface drag coefficient at the reference height z_{ref} above the sea surface, such that:

$$u_* = \sqrt{C_d(z_{ref})} V_{1hr}(z_{ref})$$

$$z_0 = 10 \times \exp\left(-\frac{k}{\sqrt{C_d(z_{ref})}}\right)$$

where the surface drag coefficient $C_d(z_{ref})$ may be calculated by:

$$C_d(z_{ref}) = [0.49 + 0.065 V_{1hr}(z_{ref})] \times 10^{-3}$$

The values of $C_d(z_{ref})$ and z_0 determined by the equations given above will increase along with the mean wind speed. However, it is found that the $C_d(z_{ref})$ levels off for the wind speed $V_{1hr}(z_{ref})$ exceeding approximately 22 m/s ~ 30 m/s (43.8 knots ~ 58.3 knots) and may even decrease slightly at very high wind speeds, probably due to wave breaking and increasingly frothy sea surface. The maximum $C_d(z_{ref})$ over the open ocean is estimated in the range of 0.0019 ~ 0.0025 depending on the size of the tropical cyclone and the maximum wind speed. As a results, the surface roughness length z_0 over the open ocean is approximately in the range of 0.001 m ~ 0.0034 m (0.0033 ft ~ 0.011 ft). For typical tropical cyclones over the open ocean, the following limiting value should be applied:

$$C_d(z_{ref}) \leq 0.0023$$

5 Standard Deviation of Tropical Cyclone Wind Speed and Turbulence Intensity

The standard deviation of instantaneous tropical cyclone wind speed within one hour time duration is determined using the formulation suggested in ESDU 83045 *Strong Winds in the Atmospheric Boundary Layer, Part 2: Discrete Gust Speeds* (2002), in conjunction with the friction velocity u_* and the surface roughness length z_0 determined in accordance with this Appendix (see A2-1/3).

$$\sigma_u(z) = \frac{7.5\eta \left[0.538 + 0.09 \ln \left(\frac{z}{z_0} \right) \right] \eta^{16}}{1 + 0.156 \ln \left(\frac{u_*}{f z_0} \right)} u_*$$

Where

- $\sigma_u(z)$ = standard deviation of the instantaneous longitudinal wind speed at the height z above the sea surface, in m/s (ft/s)
- η = scaling parameter, (dimensionless)
- $$= 1 - \frac{6fz}{u_*}$$
- f = Coriolis parameter, in rad/s
- $$= 1.458 \times 10^{-4} \sin \phi$$
- ϕ = local angle of latitude, in degrees

The turbulence intensity, defined as the standard deviation of the wind speed divided by the mean wind speed at the same height, is given by:

$$I_u(z) = \frac{\sigma_u(z)}{V_{1hr}(z)} = \frac{\sigma_u(z)}{u_*} \frac{u_*}{V_{1hr}(z)}$$

$$= \frac{7.5\eta \left[0.538 + 0.09 \ln \left(\frac{z}{z_0} \right) \right] \eta^{16}}{1 + 0.156 \ln \left(\frac{u_*}{f z_0} \right)} \frac{k}{\ln \left(\frac{z}{z_0} \right)}$$

where the friction velocity u_* can be determined either by the surface wind profile, or the relation derived from surface shear stress.

The turbulence intensity $I_u(z)$ is weakly dependent on the mean wind speed and, for strong winds, can be considered approximately independent of the mean wind speed.

7 Tropical Cyclone Wind Gust Factor

The longitudinal gust wind speed, $V_t(z)$, which represents the maximum value of longitudinal wind speed occurring in a given observation period averaged over a shorter period, is related to the mean longitudinal wind speed in the same observation period through the gust factor as defined below:

$$V_t(z) = G(z, \tau, T_0) V_{to}(z)$$

$$= [1 + g(z, \tau, T_0) I_u(z)] V_{to}(z)$$

where

$V_{\tau}(z)$	=	longitudinal gust wind speed at the height z above the sea surface with averaging time duration of τ , in m/s (ft/s)
$V_{T_0}(z)$	=	mean longitudinal wind speed at the height z above the sea surface in a given reference observation time period T_0 , in m/s (ft/s)
$G(z, \tau, T_0)$	=	gust factor, (dimensionless)
$g(z, \tau, T_0)$	=	peak factor, (dimensionless)
z	=	height above the sea surface, in m (ft)
τ	=	gust averaging time duration, in seconds
T_0	=	reference observation time duration ($T_0 > \tau$), in seconds
$I_u(z)$	=	turbulence intensity at the height z above the sea surface, (dimensionless)

For the one hour observation time period, the peak factor can be approximately calculated by:

$$g(z, \tau, T_0 = 3600s) = \frac{\sigma_u(z, \tau, T_0 = 3600s)}{\sigma_u(z)} \left[\sqrt{2 \ln(3600v)} + \frac{0.577}{\sqrt{2 \ln(3600v)}} \right]$$

$$\frac{\sigma_u(z, \tau, T_0 = 3600s)}{\sigma_u(z)} = 1 - 0.193 \left(\frac{T_u}{\tau} + 0.1 \right)^{-0.68}$$

where

$\sigma_u(z, \tau, T_0)$	=	standard deviation of the wind speed at the height z above the sea surface with averaging time duration of τ observed during a period of T_0 , in m/s (ft/s)
$\sigma_u(z)$	=	standard deviation of the instantaneous wind speed at the height z above the sea surface, in m/s (ft/s)
v	=	zero up-crossing frequency of wind speed, in Hz
	=	$\left[0.007 + 0.213(T_u/\tau)^{0.654} \right] / T_u$
T_u	=	longitudinal integral length time scale, in seconds
	=	$3.12z^{0.2}$

The gust factor associated with one hour reference time period is relate to the peak factor through:

$$G(z, \tau, T_0 = 3600s) = 1 - g(z, \tau, T_0 = 3600s)I_u(z)$$

As an example, the maximum 10-minute mean wind speed observed in a one hour time duration can be obtained by:

$$V_{10min}(z) = G(z, \tau = 600s, T_0 = 3600s)V_{1hr}(z)$$

For $z = z_{ref} = 10$ m (33 ft) above the sea surface,

$$V_{10min}(z_{ref}) = 1.03V_{1hr}(z_{ref})$$

For a reference time period $T_0 (> \tau)$ smaller than 3600 s, the gust factor can be calculated by:

$$G(z, \tau, T_0) = [0.2193 \ln(\log_{10} T_0) - 0.7242] \times G(z, \tau, T_0 = 3600s)$$

APPENDIX 3

Fatigue Analysis for the Support Structures

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APPENDIX 3

Fatigue Analysis for the Support Structures

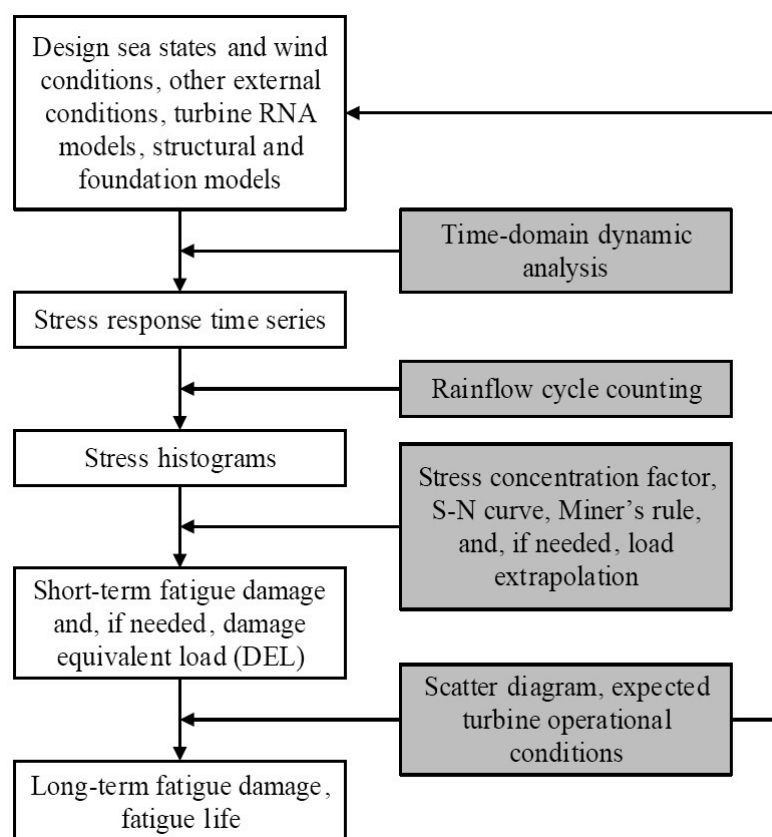
SECTION 1

Fatigue Analysis for the Support Structures

1 General (1 July 2020)

Spectral fatigue analysis method is widely used for designing fixed offshore platforms in the offshore oil and gas industry. A basic assumption of the spectral fatigue analysis method is that the offshore structure acts linearly in response to wave loading so as to the structural responses are proportional to the wave height through the transfer functions. For bottom-founded offshore wind turbines, however, such assumption may no longer be valid when aerodynamic loads become the primary source of fatigue damage in the turbine Support Structure. Such aerodynamic loads are mostly the result of nonlinear responses of the turbine RNA and are determined by nonlinear aeroelastic responses of the blades and actions of the turbine control and protection systems. Time-domain fatigue analysis method is therefore considered as a more rational approach and should in general be employed in the design of the turbine support structures. A3-1/Figure 1 depicts a typical flowchart for the time-domain fatigue analysis for bottom-founded offshore wind turbine support structures.

FIGURE 1
Flowchart of Time Domain Fatigue Analysis Method (1 July 2020)



3 Scatter Diagram Lumping

One of the main difficulties of using the time-domain fatigue analysis method in the practical design arises from the time consuming effort of numerical simulations in the time domain. This could be a costly process in particular for those support structures having complex geometries and connections, such as jacket-type support structures. The number of simulations are proportional to the number of combinations of wind, sea state and direction as defined in the scatter diagram. For a given direction, a typical scatter diagram for the fatigue analysis of a turbine support structure includes three variables [i.e., the significant wave height (H_s), the wave peak period or zero up-crossing period (T_p or T_z), and the hub-height wind speed (V_{hub})]. In order to reduce the computational effort, the scatter diagram lumping method may be used to reduce the total number of combinations of sea state and wind speed. The hub height wind speed bin and directionality should be verified in accordance with the requirement of 4-2/3. In general, the hub height wind speed bin should not be lumped.

With the assumption that the scatter diagram is expressed in the form of a series of T_p vs. H_s tables for a given hub height wind speed at a given direction, the main steps of the suggested scatter diagram lumping method are:

- i) Divide a scatter diagram into up to five lumping partitions depending on the range of structural natural period: lumping partition A of the scatter diagram with associated wave period near the first structural nature period; lumping partition B and C of the scatter diagram with associated wave periods adjacent to structural natural period within the range of plus and minus 2 seconds, respectively; and lumping partition D and E of scatter diagram with associated wave periods outside the range of the structural natural period plus and minus 2 seconds, respectively. For a typical bottom-founded support structure with the first natural period around 3 seconds and a

scatter diagram with a lower bound wave period of 3 seconds, there would be only three partitions (i.e., A, B and D).

- ii) Calculate the equivalent significant wave height and the equivalent wave period for each partition using the following equations. As a result, a full T_p - H_s scatter diagram can be condensed to no more than 5 equivalent sea states.

$$(H_s)_{eq} = \left[\frac{\sum_{i=1}^N p_i (H_s)_i^m}{\sum_{i=1}^N p_i} \right]^{\frac{1}{m}}$$

$$(T_p)_{eq} = \left[\frac{\sum_{i=1}^N p_i}{\sum_{i=1}^N p_i / (T_p)_i} \right]$$

where

- $(H_s)_{eq}$ = equivalent significant wave height, in m (ft)
- $(H_s)_i$ = significant wave height of sea state i , in m (ft)
- $(T_p)_{eq}$ = equivalent wave peak period, in seconds
- $(T_p)_i$ = wave peak period of sea state i , in seconds
- p_i = occurrence number or probability of sea state i
- m = slope of a selected S-N curve (the smaller slope in the case of a two-segment S-N curve)
- N = number of non-zero entries of p_i in a given partition of the scatter diagram

APPENDIX 4

Abbreviations and References

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APPENDIX 4 Abbreviations and References

SECTION 1 Abbreviations and References

1 Abbreviations (1 July 2020)

ABS	American Bureau of Shipping
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
API	American Petroleum Institute
ASTM	American Society for Testing and Materials
AWS	American Welding Society
IEC	International Electrotechnical Commission
ISO	International Organization for Standardization
NACE	National Association of Corrosion Engineers
NPD	Norwegian Petroleum Directorate

3 References (1 July 2020)

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