Updates

March 2018 consolidation includes:

- October 2015 version plus Corrigenda/Editorials
Foreword

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

The criteria applicable to floating offshore wind turbine installations are provided in the ABS Guide for Building and Classing Floating Offshore Wind Turbine Installations (FOWTI 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:

- 1-1/3 – A new optional class notion for the Rotor-Nacelle Assembly (RNA) is offered.
- 1-2/9 – More specific requirements were added regarding the number of turbines subject to commissioning surveys as well as the scope of commissioning survey. These requirements are adapted from the current FOWTI Guide and are in line with IEC 61400-3.
- 3-6/15 – Guidance is provided on how the site assessment should be done with regard to electric network conditions.
- 4-2/3 – Further clarification for the return period of the design storms in tropical cyclone prone areas is added.
- 4-2/Table 1 – Additional requirements are provided for considering the directionality of wind and wave that may cause misaligned wave and wind conditions. A new design load case (new DLC 8.2) is added to account for possible fatigue damage during the temporary phase.
- Appendix 2 – A new Appendix is added detailing the tropical cyclone wind model developed based on the measurements of tropical cyclone wind in the past 20 years.
- Appendix 3 – A new Appendix is added that provides guidance on fatigue analysis of support structures.

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 TURBINE INSTALLATIONS

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# Chapter 1: Classification and Surveys

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CHAPTER 1 Classification and Surveys

SECTION 1 Scope and Conditions of Classification

1 Classification

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 Turbine Installations are provided in the following Subsections.

3 Classification Symbols and Notations (15 January 2013)

(1 October 2015) The following class notations are designated for the Bottom-founded Offshore Wind Turbine Installation as defined in 1-4/3.3. Additionally, at the request of the Owner, the Rotor-Nacelle Assembly (RNA) may be included in the classification subject to the ABS Type Approval requirements of the ABS Rules for Conditions of Classification – Offshore Units and Structures (Part 1).

When the RNA is not included in the classification, the RNA installed on the ABS classed Bottom-founded Offshore Wind Turbine Installation is required to have a type certificate in accordance with IEC 61400-22 or other recognized standards. The RNA type certificate will be reviewed by ABS solely to verify that the information of the installed RNA is consistent with the design information, criteria and limitations considered in the classification of the Bottom-founded Offshore Wind Turbine Installation. ABS will not review or be responsible for the accuracy of the RNA type certificate. Use of the RNA which does not have an appropriate type certification or the wind turbine having a non-horizontal-axis configuration will be subject to special consideration by ABS.

3.1 Installations Built under ABS Survey

Installations 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 Installation (Bottom-Founded)

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

3.3 Installations Not Built under ABS Survey

Installations 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 Installation (Bottom-Founded)

3.5 Additional Class Notation (1 October 2015)

The RNA, as defined in 1-3/3.43, may be considered for the classification at the Owner’s request. Where the RNA are in full compliance with the requirements of the ABS Type Approval requirements of the ABS Rules for Conditions of Classification – Offshore Units and Structures (Part 1), the installation may be classed and distinguished in the Record by the optional class notation RNA.
5 Rules for Classification (15 January 2013)

5.1 Application

The requirements in this Guide are applicable to the Bottom-founded Offshore Wind Turbine Installation, as defined in 1-4/3.3. The design criteria specified in this Guide are intended for the Bottom-founded Offshore Wind Turbine Installation to achieve the normal safety class as defined in IEC 61400-3 (2009).

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, Owner, et al.

5.3 References

Reference is 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 a particular 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.

5.5 Alternatives (1 October 2015)

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 ABS’s approval. The ABS Guidance Notes on Risk Assessment Application for the Marine and Offshore Oil and Gas 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 Installation as a whole or to individual systems, subsystems or components. As appropriate, account must be given to remote hazards outside of the bounds of the system under consideration. 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 Installation as well as surveys during construction and surveys for maintenance of class.

Portions of the Bottom-founded Offshore Wind Turbine Installation 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

7 Design Documentation to be Submitted

(15 January 2013) 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 Installation. 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 Installation within an offshore wind farm, and

ii) The level of experience with conditions in an area where the Bottom-founded Offshore Wind Turbine Installation is to be located.

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

7.1 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.

7.1.1 Offshore Wind Farm Conditions (15 January 2013)

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 the information of 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.

7.1.2 Environmental Considerations (15 January 2013)

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 Installation, 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 Installation if they are not provided in other documentation.

7.1.3 Foundation Data

Reports on foundation data are to present the results 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.
Chapter 1 Classification and Surveys
Section 1 Scope and Conditions of Classification

The report is to include a listing 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 Chapter 5, Section 4.

7.1.4 Materials and Welding

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, when 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.

7.3 Design Data and Calculations

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 which may be required in association with environmental considerations and foundation data are described in 1-1/7.1.

7.3.1 Loadings

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

7.3.2 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.

7.3.3 Structural Responses

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 Support Structure of an offshore wind turbine, the applicable calculations required in 3-1/3 of the ABS Guide for Building and Classing Mobile Offshore Units are to be submitted.
7.3.4 Marine Operations (15 January 2013)
Calculations are to be submitted in compliance with Chapter 6. For structural responses resulting from the marine operations described in Chapter 6, 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.

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

7.5 Plans and Specifications (15 January 2013)
Plans or specifications depicting or describing arrangements and details of major items of the Bottom-founded Offshore Wind Turbine Installation 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 Installation.

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) Pile plans indicating arrangements, nominal sizes, thicknesses and penetration

vi) Welding details and procedures, and schedule of nondestructive testing

vii) Corrosion control systems

viii) Various information in support of novel features utilized in the design where applicable

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

7.7 Information Memorandum (15 January 2013)
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 to be installed.

ii) Site plan indicating general features at the installation site and the layout of the offshore wind farm

iii) Environmental design criteria, including the recurrence interval used to assess environmental phenomena

iv) Plans showing the general arrangement of the Support Structure and its connection to the RNA

v) Description of the safety and protective systems provided
Chapter 1 Classification and Surveys
Section 1 Scope and Conditions of Classification

vi) Description of modes of operation
vii) Listing of governmental authorities having cognizance over the offshore wind farm
viii) Listing of any novel features
ix) Brief description of any monitoring proposed and security systems installed on the Support Structure
x) Description of transportation, installation and maintenance procedures

9 Operating Manual (15 January 2013)

The Operating Manual of the Bottom-founded Offshore Wind Turbine Installation 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.

A copy of the Operating Manual is preferably to be stored onboard of the Bottom-founded Offshore Wind Turbine Installation. Alternatively, the Operating Manual may be retained by the Owner 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  Classification and Surveys

SECTION 2  Surveys During Construction, Installation, and Commissioning

1  Overview

1.1  Scope
This Section pertains to surveys during the construction and installation of the Bottom-founded Offshore Wind Turbine Installation. The requirements of 1-2/1 are to apply to all Bottom-founded Offshore Wind Turbine Installations covered by this Guide regardless of type. Additional specific requirements are contained in 1-2/3 for steel Support Structures and in 1-2/5 for concrete Support Structures.

The phases of construction covered by this Section include material manufacture, fabrication, installation and final field erection of the foundation, substructure, tower, and RNA.

1.3  Quality Control Program
A quality control program compatible with the type and size of the planned Bottom-founded Offshore Wind Turbine Installation 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 his representatives.

1.7  Identification of Materials
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 1-2/3.25 and 1-2/5.17). Such data are to be available to the Surveyor upon request.
3  **Steel Structures**

3.1  **Quality Control Program**

The quality control program (see 1-2/1.3) 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 1-2/3.3 through 1-2/3.23.

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**

Welders who will work on the Bottom-founded Offshore Wind Turbine Installation 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. In the event that 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 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 satisfied. 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 **Installation of the Support Structure (Foundation, Substructure, and Tower)**
Installation procedures are to be submitted to ABS for review and approval as described in Chapter 6. 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 Support Structure  
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 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.
3.23 Testing and Trials of a Self-Elevating Unit to be Used as the Support Structure

A Self-Elevating Mobile Offshore Unit intended to be converted to the site-dependent Support Structure 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 Chapter 6, Section 1 of the ABS Guide for Building and Classing Mobile Offshore Units.

3.25 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

The quality control program (see 1-2/1.3) 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 1-2/3.17)

The items which are to be considered for each of the topics mentioned above, except for viii), are indicated in 1-2/5.3 through 1-2/5.15.
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 stockpile.

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.

\[\begin{align*}
&i) \quad \text{Consistency} \\
&ii) \quad \text{Air content} \\
&iii) \quad \text{Density or Specific Gravity} \\
&iv) \quad \text{Strength}
\end{align*}\]

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.

Anchorages 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

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 Installation 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 Installation of the Support Structure (Foundation, Substructure and Tower)

Installation procedures are to be submitted to ABS for review and approval as described in Chapter 6. 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.
5.17 Records

Reference is made to 1-2/3.25 regarding the need to compile 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 Rotor-Nacelle Assemblies (RNAs) (15 January 2013)

The Surveyor is to witness the installation of at least one RNA per each type and the hook-up of the associated export electrical cable. Where there are more than fifty (50) RNAs of the same type, at least one RNA installation per every fifty (50) turbines of the same type and the hook-up of the associated export electrical cable are to be witnessed by the Surveyor. The selection of RNA installations to be witnessed is to reflect having Surveys at the start and end of RNA installation periods and the rate of RNA installation within an installation period.

The attending Surveyor is to verify that the RNAs to be installed are in compliance with the relevant design documents for the Bottom-founded Offshore Wind Turbine Installation. Deviations from approved design documents and plans or any incidents such as damage or overstress to the Bottom-founded Offshore Wind Turbine Installation during the installation of RNAs may require re-submittal of supporting documentation to provide an assessment of the significance of deviation and any necessary remedial actions to be taken.

9 Commissioning Surveys (1 October 2015)

The Surveyor is to observe the Bottom-Founded Offshore Wind Turbine Installation operating under various capacities and conditions. The Surveyor is to witness the commissioning of at least one (1) RNA per each type. Where there are more than fifty (50) RNAs of the same type, the commissioning of at least one 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. The overall performance of the turbine RNA is to be verified for compliance with the design parameters used in the design of the Bottom-Founded Offshore Wind Turbine Installation. Records of all these performance are to be maintained and made available to ABS.
CHAPTER 1 Classification and Surveys

SECTION 3 Surveys After Construction

1 Condition for Surveys after Construction

1.1 Damage, Failure, and Repair

1.1.1 Examination and Repair
Damage, failure, deterioration or repair to a classed Bottom-founded Offshore Wind Turbine Installation, 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

The Surveyor is to have access to a classed Bottom-founded Offshore Wind Turbine Installation at all reasonable times. The Owners or their representatives are to notify the Surveyor on all occasions when the Bottom-founded Offshore Wind Turbine Installation can be examined on site.

The Surveyor is to undertake all surveys on the classed Bottom-founded Offshore Wind Turbine Installation 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 Annual Surveys

Annual Surveys of the Bottom-founded Offshore Wind Turbine Installation 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.
1.7 Special Periodical Surveys

Special Periodical Surveys of the Bottom-founded Offshore Wind Turbine Installation 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.

1.9 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.

1.11 Reactivation Surveys

When the Bottom-founded Offshore Wind Turbine Installation 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 Installation had been maintained during that period.

1.13 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.

1.15 Alterations

No major alterations, which affect classification of the installation, are to be made to a classed Bottom-founded Offshore Wind Turbine Installation 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.

1.17 Survey of a Self-Elevating Unit to be Used as the Support Structure (15 January 2013)

A Self-Elevating Mobile Offshore Unit that has been converted to the site-dependent Support Structure 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 Chapter 6, Section 2 of the ABS Guide for Building and Classing Mobile Offshore Units.

1.19 Survey for Extension of Use

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 Installation. 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 1-3/1.7 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 Installation.

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.

1.21 Relocation of Existing Installations

Existing installations that are classed at a specified location require special consideration when relocation to a new site is proposed. The Owner is to advise ABS of the proposal to change locations addressing removal, transportation and re-installation aspects of the change. The survey requirements described in Chapter 1, Section 2 and 1-3/1.19, wherever applicable, are to be complied with in addition to engineering analyses required to justify the integrity of the installation for the design life at the new location.

3 Annual Surveys

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 Installation are to be obtained from ABS.
5 Special Periodical Surveys

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 Installation. 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 Installation 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 the Bottom-founded Offshore Wind Turbine Installation are to be obtained from ABS.

7 Continuous Surveys

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 Turbine Installations selected from the offshore wind farm. All the Special Periodical Survey requirements as specified in 1-3/5 are to be fulfilled during the Continuous Surveys. Each Bottom-founded Offshore Wind Turbine Installation 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 Turbine Installations.

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

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 Installation, but will not contribute to the strength of the Bottom-founded Offshore Wind Turbine Installation. 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**

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 at least submit 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 (15 January 2013)**

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 Installation.

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 Installation.

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 (15 January 2013)**

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 Installation. The plan is to be developed in accordance with the ABS Guide for Surveys Using Risk-Based Inspection for the Offshore Industry 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 Installation 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 1-3/1 through 1-3/17.
CHAPTER 1 Classification and Surveys

SECTION 4 Definitions

1 Types of Support Structures

1.1 Pile-Supported Structure
A structure supported by foundation elements or suction type piles driven into the sea floor

1.3 Gravity Structure
A structure supported by the foundation structure resting directly on the sea floor
The geometry and weight of foundation structure are selected to mobilize available cohesive and frictional strength components of the sea floor soil to resist loadings.

1.5 Self-Elevating Unit to be Used as the Support Structure
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 Support Structure 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 spudcan
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
The material application of structural members in the Bottom-founded Offshore Wind Turbine Installation 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 Installation (15 January 2013)
A Bottom-founded Offshore Wind Turbine Installation is a site-dependent Support Structure of an Offshore Wind Turbine attached on and supported by the sea floor. The Support Structure, which is defined in 1-4/3.51, encompasses the Tower, Substructure, and Foundation.

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

In this Guide, the Bottom-founded Offshore Wind Turbine Installation does not include wind turbine components such as nacelle, rotor, generators, and gear box.

3.5 Consultant
A consultant is any person who, through education and experience, has established credentials of professionalism and knowledge in the stated field.

3.7 Cut-In Wind Speed \( (V_{ci}) \)
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 \( (V_{cout}) \) (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 Shutdown
Rapid shutdown of the wind turbine triggered by a protection function or 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**

An Offshore Wind Turbine consists of a Rotor-Nacelle Assembly and its Support Structure, as defined in 1-4/3.43 and 1-4/3.51, 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**
A Surveyor is 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/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/Table 1 are provided only as a means of categorizing steels.

### TABLE 1
**Steel Tensile Properties**

<table>
<thead>
<tr>
<th>Group</th>
<th>Yield Strength $f_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ksi</td>
</tr>
<tr>
<td>II</td>
<td>$f_y &lt; 40$</td>
</tr>
<tr>
<td>II</td>
<td>$40 \leq f_y \leq 60$</td>
</tr>
<tr>
<td>III</td>
<td>$60 \leq f_y \leq 100$</td>
</tr>
</tbody>
</table>

Some of the typical ASTM and API steels belonging to the groups of 2-1/Table 1 are shown in 2-1/Table 2 and 2-1/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

<table>
<thead>
<tr>
<th>Group</th>
<th>Specification &amp; Grade</th>
<th>Yield Strength</th>
<th>Tensile Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ksi</td>
<td>MPa</td>
</tr>
<tr>
<td>I</td>
<td>ASTM A36</td>
<td>36</td>
<td>250</td>
</tr>
<tr>
<td></td>
<td>ASTM A131 Grade A (ABS Grade A)</td>
<td>34</td>
<td>235</td>
</tr>
<tr>
<td></td>
<td>ASTM A285 Grade C</td>
<td>30</td>
<td>205</td>
</tr>
<tr>
<td></td>
<td>ASTM A131 Grades B, D (ABS Grades B, D)</td>
<td>34</td>
<td>235</td>
</tr>
<tr>
<td></td>
<td>ASTM A516 Grade 65</td>
<td>35</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>ASTM A573 Grade 65</td>
<td>35</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>ASTM A709 Grade 36T2</td>
<td>36</td>
<td>250</td>
</tr>
<tr>
<td></td>
<td>ASTM A131 Grade E (ABS Grade E) (ABS Grade E)</td>
<td>34</td>
<td>235</td>
</tr>
<tr>
<td>II</td>
<td>ASTM A572 Grade 42</td>
<td>42</td>
<td>290</td>
</tr>
<tr>
<td></td>
<td>Grade 50 (to 1 in. thick)</td>
<td>50</td>
<td>345</td>
</tr>
<tr>
<td></td>
<td>(over 1 in. thick)</td>
<td>42-62</td>
<td>290-460</td>
</tr>
<tr>
<td></td>
<td>Grade 50T (to 1 in. thick)</td>
<td>50-70</td>
<td>345-515</td>
</tr>
<tr>
<td></td>
<td>(over 1 in. thick)</td>
<td>50-70</td>
<td>345-485</td>
</tr>
<tr>
<td></td>
<td>Grade 50 (to 1 in. thick)</td>
<td>50-80</td>
<td>345-550</td>
</tr>
<tr>
<td></td>
<td>(over 1 in. thick)</td>
<td>50-75</td>
<td>345-515</td>
</tr>
<tr>
<td></td>
<td>Grade 60 (to 1 in. thick)</td>
<td>46</td>
<td>315</td>
</tr>
<tr>
<td></td>
<td>(over 21/2 in. thick)</td>
<td>46</td>
<td>315</td>
</tr>
<tr>
<td></td>
<td>Grade 60 (to 1 in. thick)</td>
<td>51</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>(over 1 in. thick)</td>
<td>51</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>Grade 60 (to 1 in. thick)</td>
<td>42</td>
<td>290</td>
</tr>
<tr>
<td></td>
<td>(over 1 in. thick)</td>
<td>42</td>
<td>290</td>
</tr>
<tr>
<td></td>
<td>Grade 60 (to 1 in. thick)</td>
<td>42</td>
<td>290</td>
</tr>
<tr>
<td></td>
<td>Grades DH36, EH36 (ABS Grades DH36, EH36)</td>
<td>690</td>
<td>46</td>
</tr>
<tr>
<td></td>
<td>ASTM A537 Class 1 (21/2 in. thick)</td>
<td>50</td>
<td>345</td>
</tr>
<tr>
<td></td>
<td>ASTM A633 Grade A</td>
<td>42</td>
<td>290</td>
</tr>
<tr>
<td></td>
<td>Grades C, D</td>
<td>50</td>
<td>345</td>
</tr>
<tr>
<td></td>
<td>ASTM A678 (80) Grade A</td>
<td>50</td>
<td>345</td>
</tr>
<tr>
<td></td>
<td>ASTM A992</td>
<td>50-65</td>
<td>345-450</td>
</tr>
<tr>
<td>III</td>
<td>ASTM A537 Class 2</td>
<td>60</td>
<td>415</td>
</tr>
<tr>
<td></td>
<td>ASTM A633 Grade E</td>
<td>60</td>
<td>415</td>
</tr>
<tr>
<td></td>
<td>ASTM A678 (80) Grade B</td>
<td>60</td>
<td>415</td>
</tr>
<tr>
<td></td>
<td>API Spec 2W-Grade 60 (to 1 in. thick)</td>
<td>60-90</td>
<td>415-620</td>
</tr>
<tr>
<td></td>
<td>(over 1 in. thick)</td>
<td>60-85</td>
<td>415-585</td>
</tr>
<tr>
<td></td>
<td>API Spec 2Y-Grade 60 (to 1 in. thick)</td>
<td>60-90</td>
<td>415-620</td>
</tr>
<tr>
<td></td>
<td>(over 1 in. thick)</td>
<td>60-85</td>
<td>415-585</td>
</tr>
<tr>
<td></td>
<td>ASTM A710-Grade A Class 3 (to 2 in. thick)</td>
<td>75</td>
<td>515</td>
</tr>
<tr>
<td></td>
<td>(2 in. to 4 in. thick)</td>
<td>65</td>
<td>450</td>
</tr>
<tr>
<td></td>
<td>(over 4 in. thick)</td>
<td>60</td>
<td>415</td>
</tr>
</tbody>
</table>
TABLE 3
Structural Steel Pipes

<table>
<thead>
<tr>
<th>Group</th>
<th>Specification &amp; Grade</th>
<th>Yield Strength</th>
<th>Tensile Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ksi</td>
<td>MPa</td>
</tr>
<tr>
<td>I</td>
<td>API 5L-Grade B</td>
<td>35</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>ASTM A53 Grade B</td>
<td>35</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>ASTM A135 Grade B</td>
<td>35</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>ASTM A139 Grade B</td>
<td>35</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>ASTM A381 Grade Y35</td>
<td>35</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>ASTM A500 Grade A</td>
<td>33-39</td>
<td>230-270</td>
</tr>
<tr>
<td></td>
<td>ASTM A501</td>
<td>36</td>
<td>250</td>
</tr>
<tr>
<td></td>
<td>ASTM A106 Grade B</td>
<td>35</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>ASTM A524 (strength varies with thickness)</td>
<td>30-35</td>
<td>205-240</td>
</tr>
<tr>
<td>II</td>
<td>API 5L95 Grade X42 (2% max. cold expansion)</td>
<td>42</td>
<td>290</td>
</tr>
<tr>
<td></td>
<td>API 5L95 Grade X52 (2% max. cold expansion)</td>
<td>52</td>
<td>360</td>
</tr>
<tr>
<td></td>
<td>ASTM A500 Grade B</td>
<td>42-46</td>
<td>290-320</td>
</tr>
<tr>
<td></td>
<td>ASTM A618 Grade Ia, Ib &amp; II (to 3/4 in. thick)</td>
<td>50</td>
<td>345</td>
</tr>
<tr>
<td></td>
<td>API 5L95 Grade X52 (with SR5, SR6, or SR8)</td>
<td>52</td>
<td>360</td>
</tr>
</tbody>
</table>

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 the 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/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

<table>
<thead>
<tr>
<th>Group</th>
<th>Section Size</th>
<th>Energy Absorption (Longitudinal)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ft-lb</td>
</tr>
<tr>
<td>I</td>
<td>6 mm &lt; t &lt; 19 mm (0.25 in. &lt; t &lt; 0.75 in.)</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>t &gt; 19 mm (0.75 in.)</td>
<td>20</td>
</tr>
<tr>
<td>II, III</td>
<td>t &gt; 6 mm (0.25 in.)</td>
<td>25</td>
</tr>
</tbody>
</table>

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.

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

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.

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

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.

<table>
<thead>
<tr>
<th>Group</th>
<th>Test Temperature</th>
<th>Minimum Service Temperature (As determined by 2-1/1.7)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I, II</td>
<td>At Minimum Service Temperature</td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>Same as Note 1b</td>
<td></td>
</tr>
</tbody>
</table>

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.
TABLE 4 (continued)
Charpy Toughness Specification for Steels

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/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/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

This Subsection covers specifications for materials for concrete used in the construction of the Support Structure of an offshore wind turbine. 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 5-1/5.11):

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 not to contain more than 0.07% chlorides as Cl by weight of cement or more than 0.09% sulfates as SO₄ 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 not to 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/Table 5.
### TABLE 5

**Water-Cement Ratios and Compressive Strengths**

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

* Depending upon severity of exposure

### 3.19 Other Durability Requirements

#### 3.19.1 Cement Content

Minimum cement content is to have an adequate amount of paste for reinforcement protection and is generally 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

When severe scouring action is expected, the coarse aggregate is to be as hard as the material causing the abrasion, 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

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 1-1/7 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

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

5.1 Edge Preparation and Fitting

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 adopted 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 \(\frac{t}{2}\) or 12.5 mm (\(\frac{1}{2}\) 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 (\(\frac{1}{8}\) 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 subjected 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 subjected 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. 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.

ABS GUIDE FOR BUILDING AND CLASSING BOTTOM-FOUNDED OFFSHORE WIND TURBINE INSTALLATIONS • 2015 39
7.19 Extent of Inspection of Welds

7.19.1 General
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 determine that 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 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
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 of Hull Welds, 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 (1/4 in.) in thickness without beveling the abutting edges. Members exceeding 6.5 mm (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 (5/8 in.) in thickness, may be employed without beveling the abutting edges. Members exceeding 16 mm (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 2-2/9.1 through 2-2/9.7 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 (1/16 in.) and is not greater than 5 mm (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 (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 (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 (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 (5/16 in.) may be permitted provided that the penetration at the root is at least 1.5 mm (5/16 in.) into the members being attached and the reduced leg size is not less than 5 mm (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**

Plug welds or slot welds may be specially approved for particular 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 (1/4 in.), fillet sizes are to be equal to plate thickness but not greater than 9.5 mm (3/8 in.); for thicknesses over 13 mm (1/4 in.) to 25 mm (1 in.) fillet sizes are to be 16 mm (5/8 in.) maximum.

13 **Full Penetration Corner or Tee Joints**

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 required to assure the absence of injurious laminations prior to fabrication which could interfere with the attainment of a satisfactory welded joint.
# Environmental Conditions

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CHAPTER 3 Environmental Conditions

SECTION 1 Overview

1 General (15 January 2013)

Environmental conditions to which the Bottom-founded Offshore Wind Turbine Installation 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 Installation 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 1-1/7.1. 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 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 $V_{h,10}$ 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 $t = \frac{3600}{V_f}$ 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/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
3.3 Wind Profile (1 October 2015)

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)
- $\alpha$ = 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 storm wind conditions, the mean wind speed profile may be represented by the following logarithmic wind shear law, as recommended by API Bulletin 2INT-MET (2007). 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)

$$V(z, t_0) = V_0 \left[ 1 + C \ln \left( \frac{z}{10\phi} \right) \right]$$

- $t_0$ = reference averaging time period (1 hour), in seconds
- $= 3600$ s
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\[ C = 0.0573 \sqrt{1 + 0.15V_0 / \phi}, \text{ (dimensionless)} \]
\[ V_0 = 1\text{-hour mean wind speed at 10 m (32.8 ft) above the SWL, in m/s (ft/s)} \]
\[ I_u(z) = \text{turbulence intensity (i.e., the ratio of the wind speed standard deviation to the mean wind speed) at height } z, \text{ (dimensionless)} \]
\[ = 0.06 \left[ 1 + 0.043 \frac{V_0}{\phi} \right] \left( \frac{z}{10\phi} \right)^{-0.22} \]
\[ \phi = \text{unit conversion factor, (dimensionless)} \]
\[ = 1 \text{ when using SI units (m, m/s)} \]
\[ = 3.28 \text{ when using US Customary units (ft, ft/s)} \]

Guidance on the wind profile suitable for tropical cyclones (also known as hurricanes or typhoons) is provided in Appendix 2.

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

3.5 Wind Spectrum and Spatial Coherence (1 October 2015)

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 Subsection A1/1, or the Mann uniform shear turbulence model, as recommended in Annex B of IEC 61400-1 (2005), is to be applied.

For the tropical cyclone wind, API Bulletin 2INT-MET (2007) recommends using the NPD (Frøya) wind spectrum in conjunction with the two-point coherence function. Refer to Subsection A1/3 for the definitions.

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 (15 January 2013)

A wind condition for the design of the Bottom-founded Offshore Wind Turbine Installation is represented by a steady mean flow and associated turbulence. 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 Installation are specified in 3-2/9 and 3-2/11. The design load conditions defined in Chapter 4, Section 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)
The standard deviation of turbulence of the normal turbulence model, denoted as $\sigma_{NTM}$, 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 12.3 of IEC 61400-3 (2009) 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

(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 October 2015)
The Extreme Wind Speed Model (EWM) is defined as either a steady or 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 turbulent 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:

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

where

- $V_{hub} =$ 10-minute mean wind speed at hub height, in m/s (ft/s)
- $V_{10min,n-yr} =$ 10-minute mean wind speed at hub height with a return period of $n$ years, where $n$ is specified in 4-2/Table 1, in m/s (ft/s)
- $z =$ height above the SWL, in m (ft)
- $zhub =$ hub height above the SWL, in m (ft)

The standard deviation of longitudinal turbulent wind speed of extreme wind condition, $\sigma_1$, at hub height is to be calculated as:

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

Alternatively, 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 steady EWM, the extreme wind speeds are to be determined as follows:

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

where

- $V(z_{hub}) =$ steady extreme wind speed at hub height, in m/s (ft/s)
- $V_{3sec,n-yr} =$
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Section 2 Wind

3.2 Wind Speed

\[ V_{3\text{sec},n\text{-yr}} = \] 3-second mean wind speed at hub height with a return period of \( n \) years, where \( n \) is specified in 4-2/Table 1, in m/s (ft/s)

\[ z = \] height above the SWL, in m (ft)

\[ z_{\text{hub}} = \] hub height above the SWL, in m (ft)

Guidance on the wind profile, the turbulence intensity and the gust factor suitable for tropical cyclones (also known as hurricanes or typhoons) is provided in Appendix 2.

When using the steady EWM, constant yaw misalignment in the range of \( \pm 15^\circ \) is to be applied to account for short-term deviations from the mean wind direction.

11.3 Reduced Extreme Wind Speed Model (RWM) (1 October 2015)

The Reduced Extreme Wind Speed Model (RWM) represents a steady state extreme wind condition to be applied in combination with the extreme wave height (EWH), as defined in 3-3/7.3.

The value of extreme steady wind speed conditioned upon the extreme wave height with a given return period is to be assessed based on site-specific environmental data.

When site data are not available, the following power law wind shear is to be applied.

\[ V(z) = V(z_{\text{hub}})(z/z_{\text{hub}})^{0.11} \]

where

\[ V(z_{\text{hub}}) = \] reduced steady extreme wind speed at hub height, in m/s (ft/s) 

\[ V_{1\text{min},n\text{-yr}} = \] 1-minute mean wind speed at hub height with a return period of \( n \) years, where \( n \) is specified in 4-2/Table 1, in m/s (ft/s)

\[ z = \] height above the SWL, in m (ft)

\[ z_{\text{hub}} = \] hub height above the SWL, in m (ft)

Guidance on the wind profile and the gust factor suitable for tropical cyclones (also known as hurricanes or typhoons) is provided in Appendix 2.

Constant yaw misalignment in the range of \( \pm 15^\circ \) is to be applied to account for short-term deviations from the mean wind direction.

11.5 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:

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

where

\[ V_{3\text{sec},1\text{-yr}} = \] 3-second mean wind speed at hub height with a return period of 1 year, in m/s (ft/s)

\[ V_{hub} = \] 10-minute mean wind speed at hub height, as defined in 4-2/Table 1, in m/s (ft/s)

\[ \sigma_{NTM} = \] longitudinal turbulence standard deviation defined in 3-2/9.3, in m/s (ft/s)

\[ \Lambda_1 = \] longitudinal turbulence length scale, in m (ft)

\[ = 0.7z \quad \text{when } z \leq 60 \text{ m (196.8 ft)} \]

\[ = 42 \text{ m (137.8 ft)} \quad \text{when } z \geq 60 \text{ m (196.8 ft)} \]

\[ D = \] rotor diameter, in m (ft)
The time history of transient wind speed at height $z$ is to be defined by:

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

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

### 11.7 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:

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

where

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

### 11.9 Extreme Direction Change (EDC) (15 January 2013)

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

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

where

- $\sigma_{\text{NTM}} =$ longitudinal turbulence standard deviation defined in 3-2/9.3, in m/s (ft/s)
- $V_{\text{hub}} =$ 10-minute mean wind speed at hub height, as defined in 4-2/Table 1, in m/s (ft/s)
- $\Lambda_1 =$ longitudinal turbulence length scale, defined in 3-2/11.5, 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 ($\theta_e$). The wind speed is to follow the Normal Wind Profile Model (NWP) in 3-2/9.1.
11.11 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

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

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

\[ \theta_{cg} = \text{magnitude of direction change, in degree} \]
\[ = \begin{cases} 
180^\circ & V_{hub} \leq 4 \text{ m/s (13.12 ft/s)} \\
720^\circ(V_{hub} / \phi)^{-1} & V_{hub} > 4 \text{ m/s (13.12 ft/s)} 
\end{cases} \]

\[ V_{hub} = \text{10-minute mean wind speed at hub height, in m/s (ft/s)} \]
\[ = 10 \text{ s} \]
\[ \phi = \text{unit conversion factor, (dimensionless)} \]
\[ = 1 \text{ when using SI units (m, m/s)} \]
\[ = 3.28 \text{ when using US Customary units (ft, ft/s)} \]

11.13 Extreme Wind Shear (EWS) (15 January 2013)

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:

\[
V(z, t) = \begin{cases} 
V_{hub} \left( \frac{z}{z_{hub}} \right)^{\alpha} \pm \left( \frac{z - z_{hub}}{D} \right) \left[ 2.5\phi + 0.2\beta \sigma_{NTM} (D / \lambda_1)^{1/4} \right] \cos(\pi t / T) & 0 \leq t \leq T \\
V_{hub} \left( \frac{z}{z_{hub}} \right)^{\alpha} & \text{otherwise} 
\end{cases}
\]
The time history of transient horizontal shear is given by:

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

where

- \( V_{\text{hub}} \) = 10-minute mean wind speed at hub height, as defined in 4-2/Table 1, in m/s (ft/s)
- \( \sigma_{\text{NTM}} \) = longitudinal turbulence standard deviation defined in 3-2/9.3, in m/s (ft/s)
- \( \Lambda_1 \) = longitudinal turbulence length scale, defined in 3-2/11.5, in m (ft)
- \( z \) = height above the SWL, in m (ft)
- \( z_{\text{hub}} \) = 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)
- \( \alpha \) = 0.14, (dimensionless)
- \( \beta \) = 6.4, (dimensionless)
- \( T \) = 12 s
- \( \phi \) = 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.
CHAPTER 3 Environmental Conditions

SECTION 3 Waves

1 General (15 January 2013)

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 particular 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 in order 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. Design wave conditions may be formulated in either deterministic or probabilistic format as appropriate. 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 Installation are described in 3-3/3 through 3-3/9. 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 Wave Conditions (15 January 2013)

3.1 Normal Sea State (NSS)

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 $V_{hub}$ (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.

### 3.3 Normal Wave Height (NWH)

The Normal Wave Height (NWH) is a deterministic design wave height, $H_{NWH}$, which is defined as the expected value of the significant wave height conditioned upon a given 10-minute mean wind speed at hub height ($V_{hub}$).

A range of wave periods appropriate to each design 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. The effect of water depth at the installation site is to be considered when determining the value of wave period such that the related wave height does not exceed the breaking wave height limit.

In deep water, where the ratio of water depth to wave length is greater than 0.25, the range of wave periods may be approximated as:

$$11.1 \sqrt{\frac{E[H_s,NSS | V_{hub}]}{g}} \leq T \leq 14.3 \sqrt{\frac{E[H_s,NSS | V_{hub}]}{g}}$$

where

- $E[H_s,NSS | V_{hub}] = \text{expected value of the significant wave height of the Normal Sea State (NSS) conditioned upon a given value of } V_{hub}$, in m (ft)
- $H_{NWH} = \text{Normal Wave Height, in m (ft)}$
- $V_{hub} = \text{10-minute mean wind speed at hub height, in m/s (ft/s)}$

For shallow water sites, the upper limit of wave height and the lower limit of wave period are related to the local water depth. The following relationships may be used:

$$H_{NWH} < 0.78d \quad T > \sqrt{\frac{0.78d}{cg / 2\pi}} \left[ \text{tanh} \left( \frac{H_{NWH}}{0.78d} \right) \right]^{-1}$$

where

- $H_{NWH} = \text{normal wave height, in m (ft)}$
- $T = \text{period associated with the NWH, in seconds}$
- $d = \text{water depth water depth measured from the SWL to the sea floor, in m (ft)}$
- $g = \text{acceleration of gravity, in m/s}^2 (\text{ft/s}^2)$
- $c = 0.14$ for a regular horizontal sea floor

### 5 Severe Wave Conditions (15 January 2013)

#### 5.1 Severe Sea State (SSS)

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 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/Table 1. A series of \( V_{hub} \) is to be selected between the cut-in and cut-out wind speed for power production. As a conservative estimation, the 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.

5.3 Severe Wave Height (SWH)

The Severe Wave Height (SWH) is a deterministic design wave 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 wave height of SWH, \( H_{SWH} \), 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 \( H_{SWH}(n\text{-yr}) \) and a given \( V_{hub} \) has a specified return period of \( n \) years, where \( n \) is defined in 4-2/Table 1. A series of \( V_{hub} \) is to be selected between the cut-in and cut-out wind speed for power production. As a conservative estimation, the extreme wave height independent of wind speed may be used to approximate \( H_{SWH}(n\text{-yr}) \).

A range of wave period appropriate to each deterministic 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. The effect of water depth at the installation site is to be considered when determining the value of wave period such that the related wave height does not exceed the breaking wave height limit.

In deep water, where the ratio of water depth to wave length is greater than 0.25, the range of wave periods may be approximated as:

\[
11.1 \sqrt{\frac{E[H_{s,SSS}(n\text{-yr})V_{hub}]}{g}} \leq T \leq 14.3 \sqrt{\frac{E[H_{s,SSS}(n\text{-yr})V_{hub}]}{g}}
\]

where

- \( E[H_{s,SSS}(n\text{-yr})V_{hub}] \) = expected value of the significant wave height of the Severe Sea State conditioned upon a given value of \( V_{hub} \) and with a return period of \( n \) year, where \( n \) is specified in 4-2/Table 1, in m (ft)
- \( H_{SWH}(n\text{-yr}) \) = Severe Wave Height with a return period of \( n \) year, where \( n \) is specified in 4-2/Table 1, in m (ft)
- \( V_{hub} \) = 10-minute mean wind speed at hub height, in m/s (ft/s)
- \( g \) = acceleration of gravity, in m/s\(^2\) (ft/s\(^2\))
- \( T \) = period associated with the SWH, in seconds

For shallow water sites, the upper limit of wave height and the lower limit of wave period are related to the local water depth. The following equations may be used as an approximation.

\[
H_{SWH}(n\text{-yr}) < 0.78d \quad T > \sqrt{\frac{0.78d}{cg/2\pi\left[\tanh\left(\frac{H_{SWH}(n\text{-yr})}{0.78d}\right)\right]}}
\]

where

- \( H_{SWH}(n\text{-yr}) \) = Severe Wave Height with a return period of \( n \) year, where \( n \) is specified in 4-2/Table 1, in m (ft)
- \( T \) = period associated with the SWH, in seconds
7 Extreme Wave Conditions (15 January 2013)

7.1 Extreme Sea State (ESS)

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

The Extreme Sea State (ESS) is represented by a significant wave height, $H_{s,n\text{-yr}}$, with a return period of $n$ years, a peak spectral period and a wave direction. The values of $H_{s,n\text{-yr}}$ 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.

7.3 Extreme Wave Height (EWH)

The Extreme Wave Height (EWH) is a deterministic extreme design wave with a given return period as defined in 4-2/Table 1 for specific design load cases. The EWH with a return period of $n$ years is denoted as $H_{n\text{-yr}}$, which is to be determined from on-site measurements, hindcast data, or both for the installation site. A range of wave periods appropriate to the site-specific $H_{n\text{-yr}}$ is to be determined for the load calculation. Alternatively, in deep water where wave heights can be assumed to follow the Rayleigh distribution and the number of waves in 3 hours is approximately 1000, the extreme wave heights and the associated wave periods may be estimated by the following equations:

$$H_{n\text{-yr}} = 1.86 H_{s,n\text{-yr}}$$

$$11.1 \frac{H_{s,n\text{-yr}}}{g} \leq T \leq 14.3 \frac{H_{s,n\text{-yr}}}{g}$$

where

- $H_{n\text{-yr}}$ = extreme wave height with a return period of $n$ years, where $n$ is defined in 4-2/Table 1, in m (ft)
- $H_{s,n\text{-yr}}$ = significant wave height with a return period of $n$ years for a 3-hour reference period, in m (ft)
- $g$ = acceleration of gravity, in m/s$^2$ (ft/s$^2$)
- $T$ = period associated with the EWH, in seconds

For shallow water sites, $H_{n\text{-yr}}$ and associated wave periods are to be determined by measurements at site. In the event that such measurements are not available, $H_{n\text{-yr}}$ may be approximated conservatively by the equations given above based on the Rayleigh distribution assumption or the breaking wave height at the site, whichever is less. The upper limit of wave height $H_{n\text{-yr}}$ and the lower limit of wave period are related to the water depth and can be estimated according to 3-3/5.3.

7.5 Reduced Wave Height (RWH)

The Reduced Wave Height (RWH) is to be applied in combination with the steady Extreme Wind Speed Model (EWM), as defined in 3-2/11.1.

The RWH is defined as the Extreme Wave Height (EWH) (see 3-3/7.3) times a reduction factor, which is denoted by $r_2$ in 4-2/Table 1. The value of reduction factor is to be assessed using site-specific measurements. In the absence of site data, the RWH may be assumed as 70% of EWH or the breaking wave height at the installation site, whichever is less.
The resultant total shear forces and/or overturning moments due to the combined RWH and EWM are to be verified by those obtained by applying a combination of the EWH defined in 3-3/7.3 and the RWM defined in 3-2/11.3.

9 Breaking Waves

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

Further guidance on breaking wave hydrodynamics can be found in IEC 61400-3, Annex C.
CHAPTER 3 Environmental Conditions

SECTION 4 Currents

1 Currents (15 January 2013)

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 analogous 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:

i) Wind-generated current

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

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

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

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

\[ U_b = 2 \zeta \sqrt{g H_b} \]

where

- \( U_b \) = speed of the near shore, breaking wave induced surface current running parallel to the coastline, in m/s (ft/s)
- \( \zeta \) = sea floor slope, in rad
- \( g \) = acceleration of gravity, in m/s\(^2\) (ft/s\(^2\))
- \( H_b \) = 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 Sections 2.3.3 and 2.3.4 of API RP 2A-WSD (2007).

The current models for the design of the Bottom-founded Offshore Wind Turbine Installation are described in 3-4/3 and 3-4/5. The design load conditions defined in Chapter 4, Section 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/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 4-2/3.1x).
CHAPTER 3 Environmental Conditions

SECTION 5 Tides, Storm Surges, and Water Levels

1 General (15 January 2013)

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 air gap criterion 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/Figure 1.

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 Installation are defined in 3-5/3 and 3-5/7. The design load conditions defined in Chapter 4, Section 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) (15 January 2013)

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/Table 1. Load calculations for the strength analysis are to be performed based on the water level within the Extreme Water Level Range (EWLR) in order 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/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/Table 1
- The water level associated with the highest breaking wave load
FIGURE 1
Definition 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
CHAPTER 3 Environmental Conditions

SECTION 6 Other Conditions

1 Temperature

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 Installation.

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

Marine growth is to be considered in the design of the Bottom-founded Offshore Wind Turbine Installation. 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

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

9.1 Levels of Earthquake Conditions (15 January 2013)

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 Installation 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 Installation

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

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

The 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 Installation. 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 basis 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 mutual responses both in the ice and the structure-soil system, and this compliance is to be taken into account as applicable.

13 Lightning

The 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 Installation, provided that safety is not compromised.

15 Electric Network Conditions (1 October 2015)

The assessment of electric network conditions is to be performed in accordance with IEC 61400-3 (2009).
CHAPTER 4 Loads

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CHAPTER 4 Loads

SECTION 1 Overview

1 General (15 January 2013)

This Section pertains to the identification, definition and determination of the loads which the Bottom-founded Offshore Wind Turbine Installation 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 Installation, the types of loads described in 4-1/3 are to be considered in the design.

3 Types of Loads to be Considered

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

3.1 Dead Loads (15 January 2013)

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, etc.

iii) External hydrostatic pressure and buoyancy calculated on the basis of the SWL

iv) Static earth pressure

3.3 Live Loads (15 January 2013)

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) Weight of personnel and consumable supplies

ii) Forces exerted on the wind turbine due to lifting equipment during installation and maintenance of the wind turbine

iii) 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

iv) Loads associated with helicopter operations, where relevant

v) 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 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 Chapter 6).

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 (15 January 2013)**
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 Chapter 4, Section 2. Calculations of environmental loads are to be in accordance with Chapter 4, Section 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 (15 January 2013)

Design load conditions of the Bottom-founded Offshore Wind Turbine Installation 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.

3 Definition of Design Load Cases (DLCs) (15 January 2013)

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 Installation. 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 Installation. 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 1 – Design load cases” of IEC 61400-3 (2009), 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/Table 1. Further guidance on the analysis requirements can be found in Section 7.4 and Section 7.5.4 of IEC 61400-3 (2009).

**i)** The DLCs defined in 4-2/Table 1 are for the design of the Bottom-founded Offshore Wind Turbine Installation, which represents the site-specific Support Structure of an offshore wind turbine. As such, DLC 1.1 required by IEC 61400-3 (2009) for the calculation of ultimate loads acting on the RNA is not included in 4-2/Table 1.

**ii)** The safety factors indicated in the column entitled ‘Safety Factor’ in 4-2/Table 1 are specified in Chapter 5 of this Guide.

**iii)** The design environmental conditions in 4-2/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 (2009) 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)** (1 October 2015) 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 6.1a, 6.1b, 6.1c and DLC 6.2a and 6.2b; 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/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, as well as the scattering sensitivity of responses of the Support Structures to such variations.

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 and/or robustness assessments to evaluate the survivability of the installation using a longer return period (normally 500 years) are performed to justify such reduction.

**vi)** DLC 6.2a and 6.2b 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° 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 Wind Turbine Installation. 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 Installation.

**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/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 6.1a, 6.2a, 6.3a, 7.1a, and 8.2a, which in general require time domain dynamic simulations for the combined extreme turbulent wind and stochastic extreme waves, the simulation time duration may differ from the reference periods of wind speed and significant wave height. Two scaling factors, \( k_1 \) and \( k_2 \), are introduced in 4-2/Table 1 for the 10-minute mean wind speed and the significant wave height of a storm wave, respectively, to take this time-scale difference into account.

IEC 61400-3 (2009), Section 7.4.6 recommends to use one hour as the simulation time duration. As a result, \( k_1 = 0.95 \) for the 10-minute mean wind speed and \( k_2 = 1.09 \) for the extreme significant wave height, assuming that the reference period of extreme wave condition is 3 hours; the wave heights follow the Rayleigh distribution; and the number of waves in 3 hours is approximately 1000. The turbulence standard deviation applied in the 1-hour simulation duration is also increased by 0.2 m/s (0.66 ft/s) relative to the value associated with the 10-minute mean wind speed. Other simulation time durations can also be used along with an appropriate adjustment to the wind model and/or the wave model such that the extreme responses can be adequately estimated.

ix) Where a wind speed range is indicated in 4-2/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 (6.6 ft/s). In addition, the Rated Wind Speed (\( V_r \), see 1-4/3.39), where applicable, is to be included as one of the discrete wind speeds used in the load calculation.

x) Extreme metocean conditions in DLC 6.1a, 6.2a, 6.3a, 7.1a 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 Bulletin 2INT-MET, 2007), 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/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 8.2, 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/Table 1.

xiv) (1 October 2015) A new DLC 8.2 is added to account for the fatigue damage occurred during transport, assembly, maintenance and repair. The serial numbers of the subsequent DLCs are revised accordingly.

xv) (1 October 2015) Wind and wave directionality for DLC 1.2, 1.6a, 1.6b, 6.4, 7.2 and 8.4 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.
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 E3, E4 and E7 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) (1 October 2015) Reference is also made to API RP 2N and ISO 19906 for the alternative methods of determining static and dynamic ice loads.
<table>
<thead>
<tr>
<th>Design Condition</th>
<th>DLC</th>
<th>Wind Condition</th>
<th>Waves</th>
<th>Wind and Wave Directionality</th>
<th>Sea Currents</th>
<th>Water Level</th>
<th>Other Conditions</th>
<th>Type of Analysis</th>
<th>Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Power production</td>
<td>1.2</td>
<td>NTM, NWP</td>
<td>NSS Joint prob. distribution of $H_s$, $T_p$, $V_{hub}$</td>
<td>MIS, MUL</td>
<td>NCM</td>
<td>NWLR or ≥ MSL</td>
<td></td>
<td>F</td>
<td>FDF</td>
</tr>
<tr>
<td></td>
<td>1.3</td>
<td>ETM</td>
<td>NSS</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>MSL</td>
<td></td>
<td>S</td>
<td>S N</td>
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<td>1.4</td>
<td>ECD</td>
<td>$V_{hub} = V_r - 2$ m/s (6.6 ft/s) and $V_r + 2$ m/s (6.6 ft/s)</td>
<td>NSS (or NWH) $H_s = E[H_s</td>
<td>V_{hub}]$</td>
<td>MIS, wind direction change</td>
<td>NCM</td>
<td>MSL</td>
<td></td>
<td>S</td>
</tr>
<tr>
<td>1.5</td>
<td>EWS</td>
<td>$V_{in} ≤ V_{hub} ≤ V_{out}$</td>
<td>NSS (or NWH) $H_s = E[H_s</td>
<td>V_{hub}]$</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>MSL</td>
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</tr>
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<td>SSS</td>
<td>MIS, MUL</td>
<td>NCM</td>
<td>NWLR</td>
<td></td>
<td>$n = 50$ in general; $n = 100$ for tropical cyclone-prone sites[1]</td>
<td>S</td>
<td>S N</td>
</tr>
<tr>
<td>1.6b</td>
<td>NTM, NWP $V_{in} ≤ V_{hub} ≤ V_{out}$</td>
<td>SWH</td>
<td>MIS, MUL</td>
<td>NCM</td>
<td>NWLR</td>
<td></td>
<td>$n = 50$ in general; $n = 100$ for tropical cyclone-prone sites[1]</td>
<td>S</td>
<td>S N</td>
</tr>
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<td>2) Power production plus occurrence of fault</td>
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<td>NTM</td>
<td>NSS</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>MSL</td>
<td>Control system fault or loss of electrical network</td>
<td>S</td>
<td>S N</td>
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<td>2.2</td>
<td>NTM</td>
<td>$V_{in} ≤ V_{hub} ≤ V_{out}$</td>
<td>NSS</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>MSL</td>
<td>Protection system or preceding internal electrical fault</td>
<td>S</td>
<td>A</td>
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<tr>
<td>2.3</td>
<td>EOG</td>
<td>$V_{hub} = V_r ± 2$ m/s (6.6 ft/s) and $V_{out}$</td>
<td>NSS (or NWH) $H_s = E[H_s</td>
<td>V_{hub}]$</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>MSL</td>
<td>External or internal electrical fault including loss of electrical network</td>
<td>S</td>
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<tr>
<td>2.4</td>
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<td>NSS</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>NWLR or ≥ MSL</td>
<td>Control, protection, or electrical system faults including loss of electrical network</td>
<td>F</td>
<td>FDF</td>
</tr>
<tr>
<td>Design Condition</td>
<td>DLC</td>
<td>Wind Condition</td>
<td>Waves</td>
<td>Wind and Wave Directionality</td>
<td>Sea Currents</td>
<td>Water Level</td>
<td>Other Conditions</td>
<td>Type of Analysis</td>
<td>Safety Factor</td>
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<td>3) Start-up</td>
<td>3.1</td>
<td>NWP ( V_{in} \leq V_{hub} \leq V_{out} )</td>
<td>NSS (or NWH) ( H_s = E[H_s</td>
<td>V_{hub}] )</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>NWLR or ( \geq MSL )</td>
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<td>FDF</td>
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<td>3.2</td>
<td>EOG ( V_{hub} = V_{in}, ) ( V_{r} \pm 2 \text{ m/s (6.6 ft/s)} ) and ( V_{out} )</td>
<td>NSS (or NWH) ( H_s = E[H_s</td>
<td>V_{hub}] )</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>MSL</td>
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<td>N</td>
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<td></td>
<td>3.3</td>
<td>EDC ( V_{hub} = V_{in}, ) ( V_{r} \pm 2 \text{ m/s (6.6 ft/s)} ) and ( V_{out} )</td>
<td>NSS (or NWH) ( H_s = E[H_s</td>
<td>V_{hub}] )</td>
<td>MIS, wind direction change</td>
<td>NCM</td>
<td>MSL</td>
<td>S</td>
<td>N</td>
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<tr>
<td>4) Normal shut down</td>
<td>4.1</td>
<td>NWP ( V_{in} \leq V_{hub} \leq V_{out} )</td>
<td>NSS (or NWH) ( H_s = E[H_s</td>
<td>V_{hub}] )</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>NWLR or ( \geq MSL )</td>
<td>F</td>
<td>FDF</td>
</tr>
<tr>
<td></td>
<td>4.2</td>
<td>EOG ( V_{hub} = V_{r} \pm 2 \text{ m/s (6.6 ft/s)} ) and ( V_{out} )</td>
<td>NSS (or NWH) ( H_s = E[H_s</td>
<td>V_{hub}] )</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>MSL</td>
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<tr>
<td>5) Emergency shut down</td>
<td>5.1</td>
<td>NTM</td>
<td>NSS</td>
<td>COD, UNI</td>
<td>NCM</td>
<td>MSL</td>
<td>S</td>
<td>N</td>
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<tr>
<td>Design Condition</td>
<td>DLC</td>
<td>Wind Condition</td>
<td>Waves</td>
<td>Wind and Wave Directionality</td>
<td>Sea Currents</td>
<td>Water Level</td>
<td>Other Conditions</td>
<td>Type of Analysis</td>
<td>Safety Factor</td>
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<tr>
<td>6) Parked (standing still or idling)</td>
<td>6.1a</td>
<td>EWM Turbulent wind model $V_{hub} = k_1 V_{10min,n-yr}$</td>
<td>ESS</td>
<td></td>
<td>MIS, MUL</td>
<td>ECM $n$-yr Current</td>
<td>EWLR $n$-yr Water Level</td>
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<td></td>
<td>6.1b</td>
<td>EWM Steady wind model $V(z_{hub}) = V_{3sec,n-yr}$</td>
<td>RWH</td>
<td>$H = r_2 H_{n-yr}$</td>
<td>MIS, MUL</td>
<td>ECM $n$-yr Current</td>
<td>EWLR $n$-yr Water Level</td>
<td>$n = 50$ in general; $n = 100$ for tropical cyclone-prone sites(^3)</td>
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<tr>
<td></td>
<td>6.1c</td>
<td>RWM Steady wind model $V(z_{hub}) = V_{1min,n-yr}$</td>
<td>EWH</td>
<td>$H = H_{n-yr}$</td>
<td>MIS, MUL</td>
<td>ECM $n$-yr Current</td>
<td>EWLR $n$-yr Water Level</td>
<td>$n = 50$ in general; $n = 100$ for tropical cyclone-prone sites(^3)</td>
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<td>6.2a</td>
<td>EWM Turbulent wind model $V_{hub} = k_1 V_{10min,n-yr}$</td>
<td>ESS</td>
<td>$H = k_2 H_{n-yr}$</td>
<td>MIS, MUL</td>
<td>ECM $n$-yr Current</td>
<td>EWLR $n$-yr Water Level</td>
<td>Loss of electrical network $n = 50$ in general; $n = 100$ for tropical cyclone-prone sites(^3)</td>
<td>S</td>
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<td>6.2b</td>
<td>EWM Steady wind model $V(z_{hub}) = V_{3sec,n-yr}$</td>
<td>RWH</td>
<td>$H = r_2 H_{n-yr}$</td>
<td>MIS, MUL</td>
<td>ECM $n$-yr Current</td>
<td>EWLR $n$-yr Water Level</td>
<td>Loss of electrical network $n = 50$ in general; $n = 100$ for tropical cyclone-prone sites(^3)</td>
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<td>EWM Turbulent wind model $V_{hub} = k_1 V_{10min,n-yr}$</td>
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<td>$H = k_2 H_{n-yr}$</td>
<td>MIS, MUL</td>
<td>ECM $n$-yr Current</td>
<td>NWLR</td>
<td>Extreme yaw misalignment $n = 1$</td>
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<td></td>
<td>6.3b</td>
<td>EWM Steady wind model $V(z_{hub}) = V_{3sec,n-yr}$</td>
<td>RWH</td>
<td>$H = r_2 H_{n-yr}$</td>
<td>MIS, MUL</td>
<td>ECM $n$-yr Current</td>
<td>NWLR</td>
<td>Extreme yaw misalignment $n = 1$</td>
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<td>NCM</td>
<td>NWLR or $\geq$ MSL</td>
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### TABLE 1 (continued)

#### Design Load Cases (1 October 2015)

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<th>Design Condition</th>
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<th>Wind Condition</th>
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<th>Wind and Wave Directionality</th>
<th>Sea Currents</th>
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<th>Other Conditions</th>
<th>Type of Analysis</th>
<th>Safety Factor</th>
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</thead>
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<tr>
<td>7) Parked and fault conditions</td>
<td>7.1a</td>
<td>EWM Turbulent wind model</td>
<td>ESS</td>
<td>MIS, MUL</td>
<td>NWLR</td>
<td>n = 1</td>
<td>S</td>
<td>A</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(V_{hub} = k_1 V_{10min,n-yr})</td>
<td></td>
<td></td>
<td>ECM</td>
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<td>n-yr Current</td>
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<td>7.1b</td>
<td>EWM Steady wind model</td>
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<td>MIS, MUL</td>
<td>NWLR</td>
<td>n = 1</td>
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<td>A</td>
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<td>(V(z_{hub}) = V_{min,n-yr})</td>
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<td>ECM</td>
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<td>7.1c</td>
<td>RWM Steady wind model</td>
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<td>MIS, MUL</td>
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<td>A</td>
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<td>(V(z_{hub}) = V_{min,n-yr})</td>
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<td>(H_s, T_p, V_{hub})</td>
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<td>ECM, UNI</td>
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<td>8) Transport, assembly, maintenance and repair</td>
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<td>MIS, MUL</td>
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<td>8.3a</td>
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<td></td>
<td></td>
<td>(V_{hub} = k_1 V_{10min,n-yr})</td>
<td></td>
<td></td>
<td>ECM</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(H_s = k_2 H_{s,n-yr})</td>
<td></td>
<td></td>
<td>n-yr Current</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.3b</td>
<td>EWM Steady wind model</td>
<td>RWH</td>
<td>COD, UNI</td>
<td>NWLR</td>
<td>n = 1</td>
<td>S</td>
<td>A</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(V(z_{hub}) = V_{3sec,n-yr})</td>
<td></td>
<td></td>
<td>ECM</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(H = r_2 H_{n-yr})</td>
<td></td>
<td></td>
<td>n-yr Current</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.3c</td>
<td>RWM Steady wind model</td>
<td>EWH</td>
<td>COD, UNI</td>
<td>NWLR</td>
<td>n = 1</td>
<td>S</td>
<td>A</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(V(z_{hub}) = V_{1min,n-yr})</td>
<td></td>
<td></td>
<td>ECM</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(H = H_{n-yr})</td>
<td></td>
<td></td>
<td>n-yr Current</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.4</td>
<td>NTM</td>
<td>(V_{hub} \leq V_{1min,n-yr})</td>
<td>NSS Joint prob.</td>
<td>MIS, MUL</td>
<td>No grid during</td>
<td>NWLR or ≥ MSL</td>
<td>F</td>
<td>FDF</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(H_s, T_p, V_{hub})</td>
<td></td>
<td></td>
<td>NCM</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\text{distribution of } H_s, T_p, V_{hub})</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
- EWM: Environmental Wind Model
- RWM: Reference Wind Model
- NTM: Normal Testing Mode
- NSS: Near Shore Site
- COD: Combined Design
- UNI: Uniform
- ECM: Environmental Currents
- MIS: Mean Inflow Speed
- MUL: Mean Upwelling Longshore Currents
- NWLR: Normal Water Level Reference
- MSL: Mean Sea Level
- S: Static
- T: Transient
- A: Analytical
- FDF: Fabricator or Owner Defined
TABLE 1 (continued)
Design Load Cases (1 October 2015)

<table>
<thead>
<tr>
<th>Notes:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>(1 October 2015) 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 and/or robustness assessments using a longer return period (normally 500 years) are performed to justify such reduction.</td>
</tr>
<tr>
<td>2</td>
<td>The symbols and abbreviations used in the table are summarized as follows.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>COD</td>
<td>co-directional (aligned) wind and wave direction</td>
</tr>
<tr>
<td>DLC</td>
<td>design load case</td>
</tr>
<tr>
<td>ECD</td>
<td>extreme coherent gust with direction change (3-2/11.11)</td>
</tr>
<tr>
<td>ECM</td>
<td>extreme current model (3-4/5)</td>
</tr>
<tr>
<td>EDC</td>
<td>extreme direction change (3-2/11.9)</td>
</tr>
<tr>
<td>EOG</td>
<td>extreme operating gust (3-2/11.5)</td>
</tr>
<tr>
<td>ESS</td>
<td>extreme sea state (3-3/7.1)</td>
</tr>
<tr>
<td>ETM</td>
<td>extreme turbulence model (3-2/11.7)</td>
</tr>
<tr>
<td>EWH</td>
<td>extreme wave height (3-3/7.3)</td>
</tr>
<tr>
<td>EWLR</td>
<td>extreme water level range (3-5/5)</td>
</tr>
<tr>
<td>EWM</td>
<td>extreme wind speed model (3-2/11.1)</td>
</tr>
<tr>
<td>EWS</td>
<td>extreme wind shear (3-2/11.13)</td>
</tr>
<tr>
<td>MIS</td>
<td>misaligned wind and wave directions</td>
</tr>
<tr>
<td>MSL</td>
<td>mean sea level (3-5/Figure 1)</td>
</tr>
<tr>
<td>MUL</td>
<td>multi-directional wind and wave</td>
</tr>
<tr>
<td>NCM</td>
<td>normal current model (3-4/3)</td>
</tr>
<tr>
<td>NTM</td>
<td>normal turbulence model (3-2/9.3)</td>
</tr>
<tr>
<td>NWH</td>
<td>normal wave height (3-3/3.3)</td>
</tr>
<tr>
<td>NWLR</td>
<td>normal water level range (3-5/3)</td>
</tr>
<tr>
<td>NWP</td>
<td>normal wind profile model (3-2/9.1)</td>
</tr>
<tr>
<td>NSS</td>
<td>normal sea state (3-3/3.1)</td>
</tr>
<tr>
<td>RWH</td>
<td>reduced wave height (3-3/7.5)</td>
</tr>
<tr>
<td>RWM</td>
<td>reduced wind speed model (3-2/11.3)</td>
</tr>
<tr>
<td>SSS</td>
<td>severe sea state (3-3/5.1)</td>
</tr>
<tr>
<td>SWH</td>
<td>severe wave height (3-3/5.3)</td>
</tr>
<tr>
<td>UNI</td>
<td>uni-directional wind and wave directions</td>
</tr>
<tr>
<td>F</td>
<td>fatigue (4-2/3)</td>
</tr>
<tr>
<td>S</td>
<td>strength (4-2/3)</td>
</tr>
<tr>
<td>N</td>
<td>normal (4-2/3)</td>
</tr>
<tr>
<td>A</td>
<td>abnormal (4-2/3)</td>
</tr>
<tr>
<td>T</td>
<td>transport, assembly, maintenance and repair (4-2/3)</td>
</tr>
<tr>
<td>FDF</td>
<td>fatigue design factor (Chapter 5)</td>
</tr>
<tr>
<td>H</td>
<td>deterministic design wave height</td>
</tr>
<tr>
<td>HS</td>
<td>significant wave height</td>
</tr>
<tr>
<td>HS,n-yr</td>
<td>significant wave height of the extreme sea state with a return period of n years (3-3/7)</td>
</tr>
<tr>
<td>Hyn-yr</td>
<td>extreme wave height with a return period of n years (3-3/7)</td>
</tr>
<tr>
<td>HS,SSS</td>
<td>significant wave height of the severe sea state with a return period of n years (3-3/5)</td>
</tr>
<tr>
<td>HS,SSS,n-yr</td>
<td>design wave height of the severe sea state with a return period of n years (3-3/5)</td>
</tr>
<tr>
<td>k1</td>
<td>simulation time scaling factors for 10-minute mean wind speed (4-2/3)</td>
</tr>
<tr>
<td>k2</td>
<td>simulation time scaling factors for significant wave height (4-2/3)</td>
</tr>
<tr>
<td>n</td>
<td>return period of an extreme environmental condition (4-2/3)</td>
</tr>
<tr>
<td>r2</td>
<td>reduction factor for extreme wave height (3-3/7.5)</td>
</tr>
<tr>
<td>Tp</td>
<td>peak period of wave spectrum</td>
</tr>
<tr>
<td>V1min,n-yr</td>
<td>1-minute mean wind speed at hub height with a return period of n years (3-2/11)</td>
</tr>
<tr>
<td>V10min,n-yr</td>
<td>10-minute mean wind speed at hub height with a return period of n years (3-2/11)</td>
</tr>
<tr>
<td>V3sec,n-yr</td>
<td>3-second mean wind speed at hub height with a return period of n years (3-2/11)</td>
</tr>
<tr>
<td>Vhub</td>
<td>10-minute mean wind speed at hub height</td>
</tr>
<tr>
<td>Vzhub</td>
<td>steady wind speed at hub height, zhub</td>
</tr>
<tr>
<td>Vin</td>
<td>cut-in wind speed (1-4/3.7)</td>
</tr>
<tr>
<td>Vout</td>
<td>cut-out wind speed (1-4/3.9)</td>
</tr>
<tr>
<td>Vr</td>
<td>rated wind speed (1-4/3.39)</td>
</tr>
<tr>
<td>Vr ± 2 m/s</td>
<td>sensitivity to the wind speeds in the given range (4-2/3)</td>
</tr>
<tr>
<td>Design Condition</td>
<td>DLC</td>
</tr>
<tr>
<td>------------------</td>
<td>-----</td>
</tr>
<tr>
<td>Power Production</td>
<td>E1</td>
</tr>
<tr>
<td></td>
<td>E2</td>
</tr>
<tr>
<td></td>
<td>E3</td>
</tr>
<tr>
<td></td>
<td>E4</td>
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<td>E5</td>
</tr>
<tr>
<td>Parked</td>
<td>E6</td>
</tr>
<tr>
<td></td>
<td>E7</td>
</tr>
</tbody>
</table>

**Notes:**

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

- \( h_{ice} \) ice thickness
- \( h_{ice, 50\text{-yr}} \) extreme ice thickness with a return period of 50 years
- \( h_{ice, m} \) long term mean value of the annual maximum ice thickness
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 Chapter 3, Section 2 and the DLCs in Chapter 4, Section 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:

$$F_w = \frac{1}{2} \rho g C_s A V^2$$

where

- $F_w$ = wind drag load, in N (lb)
- $\rho$ = weight density of air, in N/m$^3$ (lb/ft$^3$)
- $g$ = acceleration of gravitation, in m/s$^2$ (ft/s$^2$)
- $C_s$ = shape coefficient
\[ A = \text{projected area of member on a plane normal to the direction of the considered force, in m}^2 \text{ (ft}^2) \]

\[ V = \text{wind speed at a given elevation above the SWL, in m/s (ft/s)} \]

In the absence of experimental data, values in 4-3/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)**

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

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/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 (15 January 2013)**

A sufficient range of realistic wave periods and wave crest positions relative to the structure are to be investigated to determine 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

\[ F \quad \text{hydrodynamic force vector per unit length along the member, acting normal to the axis of the member, in N/m (lb/ft)} \]
\[ F_D \quad \text{drag force vector per unit length, in N/m (lb/ft)} \]
\[ F_I \quad \text{inertia force vector per unit length, in N/m (lb/ft)} \]

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

\[ F_D = \left( \frac{\rho}{2g} \right) D C_D |u_n| \]

where

\[ \rho = \text{weight density of water, in N/m}^3 \text{ (lb/ft}^3) \]
\[ g = \text{acceleration of gravitation, in m/s}^2 \text{ (ft/s}^2) \]
\[ D = \text{projected width of the member in the direction of the cross-flow component of velocity (in the case of a circular cylinder, } D \text{ denotes the diameter), in m (ft)} \]
\[ C_D = \text{drag coefficient} \]
\[ u_n = \text{component of the fluid velocity vector normal to the axis of the member, in m/s (ft/s)} \]
\[ |u_n| = \text{absolute value of } u_n, \text{ in m/s (ft/s)} \]

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

\[ F_I = \left( \frac{\rho}{g} \right) \left( \pi D^2/4 \right) C_M a_n \]

where

\[ C_M = \text{inertia coefficient based on the displaced mass of fluid per unit length} \]
\[ a_n = \text{component of the fluid acceleration vector normal to the axis of the member, in m/s}^2 \text{ (ft/s}^2) \]

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.

\[ F_D = \left( \frac{\rho}{2g} \right) D C_D (u_n - \bar{u}_n) |u_n - \bar{u}_n| \]
\[ F_I = \left( \frac{\rho}{g} \right) \left( \pi D^2/4 \right) a_n + \left( \frac{\rho}{g} \right) \left( \pi D^2/4 \right) C_m (a_n - \bar{a}_n) \]

where

\[ \bar{u}_n = \text{component of the velocity vector of the structural member normal to its axis, in m/s (ft/s)} \]
\[ \bar{a}_n = \text{component of the acceleration vector of the structural member normal to its axis, in m/s}^2 \text{ (ft/s}^2) \]
\[ C_m = \text{added mass coefficient} \]
\[ = C_M - 1 \]

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-Krylov 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 wave length 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 needs 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, Annex C and D.

7 Current Loads

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.

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.

\[ F_L = \frac{\rho g}{2} AC_L U^2 \]
\[ F_D = \frac{\rho g}{2} AC_D U^2 \]

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)
- \( \rho \) = 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**

For the Bottom-founded Offshore Wind Turbine Installation 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 Installation. 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) 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 manner in which the time histories are used is 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** *(15 January 2013)*

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 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.
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CHAPTER 5 Structure and Foundation Design

SECTION 1 General Design Requirements

1 Overview (15 January 2013)

This Section provides general requirements to be applied to the structural and foundation design of the Bottom-founded Offshore Wind Turbine Installation. The criteria in Sections 2 through 4 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 the references in Chapter 5 are made to 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 1-1/7.

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.

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**

The design life of the Bottom-founded Offshore Wind Turbine Installation 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 1-3/1.19 and additional engineering analyses.

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/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 October 2015)**

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 in 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**

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 1-4/3.45. 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 Support Structures

7.1 General
Specific design considerations listed in this Subsection are to be taken into account for particular types of Support Structure 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 Guide.

7.3 Pile-Supported Steel Structures
Consideration is to be given to the following effects, as appropriate to the Support Structure:

i) The soil-pile interaction and the loads imposed on the Support Structure 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 Support Structure are to be checked to verify that the Support Structure 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
Consideration is to be given to the following effects, as appropriate to the Support Structure:

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 sulfate 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 Support Structure

A self-elevating mobile offshore unit converted to the site dependent Support Structure of an offshore wind turbine is to be designed in accordance with this Guide in conjunction with the ABS Guide 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 spudcans, 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 Support Structure, 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

(15 January 2013) The requirements of this Section are to be applied in the design and analysis of the steel Support Structure of an offshore wind turbine. Chapter 5, Section 1 is to be consulted regarding general design requirements. Items to be considered in the design of welded connections are specified in Chapter 2, Section 2.

1.1 Materials

The requirements of this Section are specified for the Support Structure 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

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 Support Structure 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

In the design of the Support Structure, 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 Support Structure 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 (15 January 2013)

The steel Support Structure 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 Support Structure are to satisfy the acceptance criteria in 5-2/9 through 5-2/23.

5 Loading Conditions (15 January 2013)

Loading conditions for the structural analysis and design of the steel Support Structure are to be determined in accordance with 5-1/5.1. The Design Load Cases (DLCs) in 4-2/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.
7 Structural Analysis (15 January 2013)

The following is general guidance on choosing appropriate approaches to perform structural analyses for the Support Structure 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 Support Structure.

iii) The interaction among environmental loadings and between external loads and structural responses are to be assessed by applying environmental loads simultaneously to the Support Structure. 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 Support Structure. 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 requirements 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.

9 Strength Design Criteria (15 January 2013)

9.1 General

The design of the steel Support Structure 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 of the same structural component of the Support Structure.

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

9.3 Working Stress Design (WSD) Approach

When the strength design of the steel Support Structure is based on the WSD approach, the design acceptance criteria are expressed in terms of appropriate allowable stresses. Linear, elastic methods can be employed in the analysis of the Support Structure 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.

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.

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:

\[ F_{\text{allowable}} = \frac{F_y}{F.S.} \]

where

- \( F_{\text{allowable}} \) = 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)
  - \( = 1.5 \) for axial or bending stress
  - \( = 2.5 \) for shear stress

- For the abnormal design conditions (designated ‘A’ in the column entitled ‘Safety Factor’ in 4-2/Table 1 and 4-2/Table 2)
  - \( = 1.25 \) 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/Table 1)
  - \( = 1.67 \) for axial or bending stress
  - \( = 2.75 \) for shear stress

9.3.2 Buckling Strength of Structural Members Subjected to a Single Action

Buckling is to be considered for a structural element under 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:

\[ F_{\text{allowable}} = \frac{F_{cr}}{F.S.} \]

where

- \( F_{\text{allowable}} \) = 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)
\[ \psi = 1.25/\psi \]
• For the design load conditions involving the transport, assembly on site, maintenance and repair operations (DLC 8.1 in 4-2/Table 1)
\[ \psi = 1.67/\psi \]
\[ \psi = \text{adjustment factor, as defined in Subsection 1/11 of the ABS Guide for Buckling and Ultimate Strength Assessment for Offshore Structures} \]

9.3.3 Structural Members Subjected 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:
\[ F_{\text{allowable}} = \frac{F_y}{F.S.} \]
where
\[ F_{\text{allowable}} = \text{allowable stress} \]
\[ F_y = \text{specified minimum yield strength, as defined in the ABS Rules for Materials and Welding (Part 2)} \]
\[ F.S. = \text{factor of safety} \]
• For the normal design conditions (designated ‘N’ in the column entitled ‘Safety Factor’ in 4-2/Table 1 and 4-2/Table 2)
\[ \psi = 1.33 \]
• For the abnormal design conditions (designated ‘A’ in the column entitled ‘Safety Factor’ in 4-2/Table 1 and 4-2/Table 2)
\[ \psi = 1.11 \]
• For the design load conditions involving the transport, assembly on site, maintenance and repair operations (DLC 8.1 in 4-2/Table 1)
\[ \psi = 1.5 \]

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

The load combination is to be in accordance with 5-2/5 in general. The partial safety factors ($\gamma$) for loads, as specified in 5-2/Table 1 (adapted from IEC 61400-3 (2009)) are to be applied to the aggregate dead loads, maximum live loads and design environmental loads.
When 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 to this load instead of those defined in 5-2/Table 1. When 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**

<table>
<thead>
<tr>
<th>Normal (N)</th>
<th>Abnormal (A)</th>
<th>Transport, Assembly, Maintenance and Repair (T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.35</td>
<td>1.1</td>
<td>1.5</td>
</tr>
</tbody>
</table>

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

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.

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 a flat plate or a 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 $\psi$ 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.

11 Structural Response to Earthquake Loads (15 January 2013)

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. Reference is made to 3-6/9 and 4-3/11 for the definitions of the Strength Level and Ductility Level earthquakes as well as the 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 (15 January 2013)

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 Installation.

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 be structural members and connections that are difficult to inspect and repair once the Bottom-founded Offshore Wind Turbine Installation 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 Installation times the safety factors for fatigue life [i.e., fatigue design factors (FDFs)] as specified in 5-2/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/Table 1 and, if applicable, 4-2/Table 2, where ‘F’ in the column titled ‘Type of Analysis’ designates the fatigue assessment. Loading history of the Support Structure 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>
<thead>
<tr>
<th>Importance</th>
<th>Inspectable and Repairable</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Yes</td>
</tr>
<tr>
<td>Non-Critical</td>
<td>1</td>
</tr>
<tr>
<td>Critical</td>
<td>3</td>
</tr>
</tbody>
</table>

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.
Chapter 5  Structure and Foundation Design
Section 2  Steel Structures

17  Structural Connections (15 January 2013)

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 formation 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 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 connections expected to resist bending moments, their strength is 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.

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 (15 January 2013)

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

23  Local Structures

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

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

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

1.1  Materials

Unless otherwise specified, the requirements of this Section are intended for the Support Structures constructed of materials manufactured and having properties as specified in Chapter 2, Section 1. Use of materials having properties differing from those specified in Chapter 2, Section 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 (15 January 2013)

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

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, $f'_m$, by the appropriate strength reduction factor, $\phi$ (i.e., design strength $= \phi f'_m$). The material strength, $f'_m$, 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
Design considerations for concrete Support Structures 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
The strength of the concrete Support Structure 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 Support Structure 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 (15 January 2013)
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/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.

For the design of the concrete Support Structure of an offshore wind turbine, the partial safety factors for loads, as specified in 5-2/Table 1, are to be applied to the aggregate dead loads, maximum live loads, deformation loads and design environmental loads.

When a dead load, live load or deformation load is considered as a favorable load that relieves total load responses, a partial safety factor of 0.9 is to be applied to this load instead of those defined in 5-2/Table 1. When a live load is considered a favorable load, the minimum value of this live load is to be used in the load combination.

For strength evaluation, the effects of deformation load may be ignored provided adequate ductility is demonstrated.

While the critical design loadings are to be identified from the load combinations described above, 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, $\phi$.

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, \( \varphi = 0.70 \)

• For other reinforced members (excluding slabs and shells), \( \varphi = 0.65 \)

The values given in the above for two types of members may be increased linearly to 0.9 as \( \varphi P_u \) decreases from 0.1 \( f'_c A_g \) or \( \varphi P_b \), whichever is smaller, to zero, where

\[
\begin{align*}
    f'_c &= \text{specified compression strength of concrete} \\
    A_g &= \text{gross area of section} \\
    P_u &= \text{axial design load in compression member} \\
    P_b &= \text{axial load capacity assuming simultaneous occurrence of the ultimate strain of concrete and yielding of tension steel}
\end{align*}
\]

• For slabs and shells, \( \varphi = 0.70 \)

iii) In the case of shear and torsion, \( \varphi = 0.75 \)

iv) In the case of bearing on concrete, \( \varphi = 0.65 \), except for post-tensioning anchorage bearing. For bearing on concrete in post-tension anchorage, \( \varphi = 0.85 \).

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 \( (\varphi) \) and the material factors \( (c_M) \) are not to be used simultaneously.

5.7 Fatigue

The fatigue strength of the concrete Support Structure of an offshore wind turbine is considered satisfactory if under the unfactored fatigue loads (i.e., \( c_k = 1 \)) the following conditions are satisfied.

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 0.5 \( f'_c \) 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/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 subjected 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 Support Structure, the safety factors for fatigue life [i.e., fatigue design factors (FDFs)] are to be in accordance with 5-2/Table 2, except that the calculated fatigue life is to be at least twice the design life of the Bottom-founded Offshore Wind Turbine Installation. 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

The serviceability of the concrete Support Structure 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, $\phi$, and partial safety factors for loads, $c_i$, are to be taken as 1.0. The unfactored $(c_i =1.0)$ load combination of most unfavorable dead loads, live loads and deformation 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/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/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.

1) The reinforcing steel stresses are to be in accordance with 5-3/5.9.1.

2) The compression zone is to extend over 25% of the wall thickness or 205 mm (8 in), whichever is less.

3) There is no membrane tensile stress unless other construction arrangements are made, such as the use of special barriers to prevent leakage.

### TABLE 1

<table>
<thead>
<tr>
<th>Stage</th>
<th>Loading</th>
<th>Allowable Stress, MPa (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Reinforcing Steel, $f_s$</td>
</tr>
<tr>
<td>Construction: where cracking during construction would be detrimental to the completed structure</td>
<td>All loads on the structure during construction</td>
<td>160 (23.0)</td>
</tr>
<tr>
<td>Construction: where cracking during construction is not detrimental to the completed structure</td>
<td>All loads on the structure during construction</td>
<td>210 (30.0) or $0.6f_y$, whichever is less</td>
</tr>
<tr>
<td>Transportation and installation</td>
<td>All loads on the structure during transportation and installation</td>
<td>160 (23.0)</td>
</tr>
<tr>
<td>At offshore site</td>
<td>Dead and live loads plus design environmental loads</td>
<td>0.8 $f_y$</td>
</tr>
</tbody>
</table>

$f_y$ = 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 (15 January 2013)

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 Support Structure. 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.

When 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

For early-strength concrete, the age for the tests for $f'_c$ may be determined on the basis of 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/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/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 $200 \times 10^3$ MPa ($29 \times 10^3$ 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/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

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 devoted 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 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

The applicable requirements of ACI 318 or their equivalent are to be complied with in the analysis and design of members subjected to shear or torsion or to combined shear and torsion.

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/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

$E_s = 200 \times 10^3$ MPa ($29 \times 10^3$ ksi)
7.13 Seismic Analysis

7.13.1 Dynamic Analysis
For the Support Structures 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 Support Structure 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 Support Structures 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 subjected 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 subjected 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.

\[ A_s = \left( f_t / f_y \right) b d_e \]

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 subjected to significant fatigue loading. Where lapped splices are used in members subjected 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 subjected 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 subjected to tension during operation, through-slab confinement reinforcement is to be considered at the splices. Where longitudinal bars are subjected 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 in order 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 dislocation 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 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

Consideration is to be given to preventing 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

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 Support Structure 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 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 concrete where necessary. The aggregates and water used in the mix are to be free from snow, ice and frost. The temperature of 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 concrete, caused by the heat of hydration of cement, is to be controlled to prevent steep temperature stress gradients which could cause cracking of 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 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 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

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 entry 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. 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

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 Installation. 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

The design data and documentation to be submitted for ABS review are summarized in 1-1/7. 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
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 Installation.

5.3 Sea Floor Survey
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 Installation.
i) Soundings or contours of the sea bed
ii) Position of bottom shapes which might affect scour
iii) 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
Data of the regional geological characteristics which can affect the design and siting of the Bottom-founded Offshore Wind Turbine Installation 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.

1) Perform unconfined compression tests on clay strata where needed to supplement field data.

2) Determine water content and Atterberg limits on selected cohesive samples.

3) 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

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 Installation.

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 consideration by ABS.

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**

Tolerable limits of deflections and rotations are to be established based on the type of the Support Structure 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 may be accomplished by lumped parameter, foundation impedance functions, or 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 subjected 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**

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 Installation are not impaired.

9 **Pile Foundations**

9.1 **General**

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 Installation.
The following is to be considered in the evaluations of the pile holding capacity.

1. Cyclic, creep and soil set-up effects on the soil strength
2. Punch through failure in layered soils
3. Installation tolerances and possible soil disturbance due to protuberances as applicable
4. Potential of soil liquefaction and its impact, if the reference site is within a seismic zone
5. Pile group effect, if applicable
6. 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 Chapter 5, Section 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 Installation.

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/Table 1. For the Design Earthquake, the factor of safety is to be specially considered.

#### TABLE 1

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Factor of Safety</th>
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<tbody>
<tr>
<td>Normal</td>
<td>2.0</td>
</tr>
<tr>
<td>Abnormal</td>
<td>1.5</td>
</tr>
</tbody>
</table>
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 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

The design method in 5-4/9.3 for 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 resist 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 is 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/Table 2. The safety factors to be applied when considering the earthquake conditions will be specially considered.

<table>
<thead>
<tr>
<th>Loading Condition</th>
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TABLE 2
Factor of Safety for Allowable Shear Strength of Soil

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.
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CHAPTER 6 Marine Operations

SECTION 1 Marine Operations

1 General (15 January 2013)

In this Guide, marine operations generally include the following activities as appropriate to the planned Bottom-founded Offshore Wind Turbine Installation.

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 Installation.

3 Documentation

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 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 6-1/5. The purpose of this report is to demonstrate that the strength and integrity of the Support Structure are not reduced or otherwise jeopardized by marine operations.

Generally, this report is to contain the following information:

i) Description of 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 the design of the Support Structure, the assumptions, calculations and results of the analyses required in 6-1/5

5 Analysis

5.1 Loads

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/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 8.2a, b, c and DLC 8.3 in 4-2/Table 1 are also to be considered in accordance with Section 7.4.8 of IEC 61400-3 (2009).
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.
APPENDIX 1 Wind Spectra and Coherence Functions

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APPENDIX 1 Wind Spectra and Coherence Functions

1 Kaimal Wind Spectrum and Exponential Coherence Model

A modified version of the Kaimal wind spectrum is provided in IEC 61400-1 (2005). The power spectral densities for the wind fluctuations in three dimensions are given as follows:

\[
\frac{f \cdot S_k(f)}{\sigma_k^2} = \frac{4fL_k / V_{\text{hub}}}{(1 + 6fL_k / V_{\text{hub}})^{5/3}}
\]

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/Figure 1)
- \(\sigma_k\) = standard deviation of turbulent wind speed component (see A1/Table 1)
- \(L_k\) = integral parameter of turbulent wind speed component (see A1/Table 1)

\[
\text{TABLE 1 Spectral Parameters for the Kaimal Model}
\]

<table>
<thead>
<tr>
<th>Wind Speed Direction</th>
<th>(k=1) (longitudinal)</th>
<th>(k=2) (lateral)</th>
<th>(k=3) (upward)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Deviation ((\sigma_l))</td>
<td>(\sigma_l)</td>
<td>(0.8\sigma_l)</td>
<td>(0.5\sigma_l)</td>
</tr>
<tr>
<td>Integral Scale ((L_l))</td>
<td>(8.1\Lambda_l)</td>
<td>(2.7\Lambda_l)</td>
<td>(0.66\Lambda_l)</td>
</tr>
</tbody>
</table>

Notes:

- \(\sigma_l\) = standard deviation of longitudinal turbulent wind speed
- \(\Lambda_l\) = 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 (2005) to account for the spatial correlation of the longitudinal wind speed:

\[
\text{Coh}(r, f) = \exp \left[ -12\sqrt{ \left( \frac{f \cdot r}{V_{\text{hub}}} \right)^2 + \left( \frac{0.12 \cdot r}{L_c} \right)^2 } \right]
\]

where

- \(\text{Coh}(r, f)\) = 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)
Appendix 1 Wind Spectra and Coherence Functions

$V_{hub} = 10$-minute mean wind speed at hub height, in m/s (ft/s)

$L_c = \text{coherence scale parameter, in m (ft)}$

$= 8.1\Lambda_1$, where $\Lambda_1$ is specified in 3-2/11.5

3 API Recommended Wind Spectrum and Coherence Model

(1 October 2015) The wind spectrum and coherence model recommended by API RP 2A-WSD (2007) and API Bulletin 2INT-MET (2007) are intended for the design of offshore structures and structural elements for which the wind load induced dynamic response needs to be considered.

3.1 Wind Spectrum

The following wind spectrum is defined for the energy density of the longitudinal wind speed fluctuations:

$$S(f) = \frac{320\phi^2 U_0^2 (z/10\phi)^{0.45}}{[1 + \frac{f^n}{\bar{f}}]^{1.5/3e}}$$

where

$S(f) = \text{spectral energy density at frequency } f$, in m$^2$s$^{-2}$/Hz (ft$^2$s$^{-2}$/Hz)

$f = \text{frequency, in Hz}$

$U_0 = \text{1-hour mean wind speed at 10 m (32.8 ft) above the SWL, in m/s (ft/s)}$

$n = 0.468$

$z = \text{height above the SWL, in m (ft)}$

$\phi = \text{unit conversion factor, (dimensionless)}$

$= 1 \text{ when using SI units (m, m/s)}$

$= 3.28 \text{ when using US Customary units (ft, ft/s)}$

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:

$$Coh(f) = \exp\left(-\frac{1}{U_0/\phi} \left[ \sum_{i=1}^{4} \frac{A_i}{\bar{A}_i} \right] \frac{1}{2} \right)$$

$$A_i = \alpha_i f^{\phi_i} \left( \frac{A_i}{\phi} \right)^{q_i} \left( \frac{\sqrt{z_1z_2}}{10\phi} \right)^{r_i}$$

where the coefficients $\alpha, p, q, r$, and the distances $A_i$ are specified in A1/Table 2, and

$Coh(f) = \text{coherence function at frequency } f$

$f = \text{frequency, in Hz}$

$U_0 = \text{1-hour mean wind speed at 10 m (32.8 ft) above the SWL, in m/s (ft/s)}$

$(x_j, y_j, z_j) = \text{spatial coordinates of two points (} j = 1, 2 \text{) where } x_j \text{ is in the longitudinal (along wind) direction, } y_j \text{ is in the lateral (across wind) direction and } z_j \text{ is the height above the SWL in the upward direction, in m (ft)}$

$\phi = \text{unit conversion factor, (dimensionless)}$

$= 1 \text{ when using SI units (m, m/s)}$

$= 3.28 \text{ when using US Customary units (ft, ft/s)}$
### TABLE 2
Coefficients and Distances for the Three-dimensional Coherence Function

<table>
<thead>
<tr>
<th>i</th>
<th>$\Delta_i$</th>
<th>$q_i$</th>
<th>$p_i$</th>
<th>$r_i$</th>
<th>$\alpha_i$</th>
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</thead>
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<tr>
<td>1</td>
<td>$</td>
<td>x_2-x_1</td>
<td>$</td>
<td>1.00</td>
<td>0.4</td>
</tr>
<tr>
<td>2</td>
<td>$</td>
<td>y_2-y_1</td>
<td>$</td>
<td>1.00</td>
<td>0.4</td>
</tr>
<tr>
<td>3</td>
<td>$</td>
<td>z_2-z_1</td>
<td>$</td>
<td>1.25</td>
<td>0.5</td>
</tr>
</tbody>
</table>
APPENDIX 2  Tropical Cyclone Wind Speed Profile, Standard Deviation, Turbulence Intensity and Gust Factor
(1 October 2015)

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3  Tropical Cyclone Wind Speed Profile .............................................. 128
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APPENDIX 2 Tropical Cyclone Wind Speed Profile, Standard Deviation, Turbulence Intensity and Gust Factor

1 Introduction

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 calculated in accordance with API RP 2MET, as adapted in 4-2/3.3 of this Guide, using the Frøya wind model for tropical cyclone-prone areas, although the model was 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 in accordance with API RP 2MET. 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 accordance with API RP 2MET.

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:

$$z_0 = \frac{\alpha u_*^2}{g} = \frac{\alpha}{g} k V_{1hr}(z) \ln \left( \frac{z}{z_0} \right)^2$$

where

$$\alpha = 0.011,$$ as suggested in IEC 61400-3, for the open ocean

$$z_{ref} = 10 \text{ m (33 ft) reference height above the sea surface, in m (ft)}$$

$$V_{1hr}(z_{ref}) = 1\text{-hour mean wind speed at the reference height } z_{ref} \text{ above the sea surface, in m/s (ft/s)}$$

$$g = \text{acceleration of gravity, in m/s}^2 (\text{ft/s}^2)$$

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 $z_{ref} = 10 \text{ m (33 ft)}$ through the surface shear stress $\tau_0 = \rho u_*^2 = \rho C_d(z_{ref}) V_{1hr}^2(z_{ref})$, where $\rho$ 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 result, 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 Subsection A2/3).

$$\sigma_f(z) = \frac{7.5\eta^{0.538 + 0.091\ln \left( \frac{z}{z_0} \right)^{0.6}}}{1 + 1.56 \ln \left( \frac{u_*}{\eta z_0} \right)} u_*$$
where

\[ \sigma_u(z) = \text{standard deviation of the instantaneous longitudinal wind speed at the height } z \text{ above the sea surface, in m/s (ft/s)} \]
\[ \eta = \text{scaling parameter, (dimensionless)} \]
\[ = \frac{6f}{u_*} \]
\[ f = \text{Coriolis parameter, in rad/s} \]
\[ = 1.458 \times 10^{-4} \sin \phi \]
\[ \phi = \text{local angle of latitude, in degrees} \]

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:

\[ I_u(z) = \frac{\sigma_u(z)}{\sqrt[1/4]{z/V_1fr(z)}} \]
\[ = \frac{7.5\eta}{1 + 0.156\ln \left( \frac{u_*}{z_0^*} \right) - \ln \left( \frac{z}{z_0} \right)} \]

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, \( V(\tau) \), 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:

\[ V(\tau) = G(z, \tau, T_0) \frac{V(\tau)}{T_0} \]
\[ = [1 + g(z, \tau, T_0) I_u(z)] \frac{V(\tau)}{T_0} \]

where
\[ V(\tau) = \text{longitudinal gust wind speed at the height } z \text{ above the sea surface with averaging time duration of } \tau, \text{ in m/s (ft/s)} \]
\[ V(\tau) = \text{mean longitudinal wind speed at the height } z \text{ above the sea surface in a given reference observation time period } T_0, \text{ in m/s (ft/s)} \]
\[ G(z, \tau, T_0) = \text{gust factor, (dimensionless)} \]
\[ g(z, \tau, T_0) = \text{peak factor, (dimensionless)} \]
\[ z = \text{height above the sea surface, in m (ft)} \]
\[ \tau = \text{gust averaging time duration, in seconds} \]
\[ T_0 = \text{reference observation time duration } (T_0 > \tau), \text{ in seconds} \]
\[ I_u(z) = \text{turbulence intensity at the height } z \text{ above the sea surface, (dimensionless)} \]
For the one hour observation time period, the peak factor can be approximately calculated by:

\[
g(z, \tau, T_0 = 3600s) = \frac{\sigma_u(z, \tau, T_0 = 3600s)}{\sigma_u(z)} \left[ \frac{\sqrt{2 \ln(3600\nu)}}{\sqrt{2 \ln(3600\nu)}} + \frac{0.577}{\sqrt{2 \ln(3600\nu)}} \right]^{0.68}
\]

\[
\sigma_u(z, \tau, T_0 = 3600s) = \sigma_u(z) + 0.193 \left( \frac{T_u}{\tau} + 0.1 \right)^{-0.68}
\]

where

\[
\sigma_u(z, \tau, T_0) = \text{standard deviation of the wind speed at the height } z \text{ above the sea surface with averaging time duration of } \tau \text{ observed during a period of } T_0, \text{ in m/s (ft/s)}
\]

\[
\sigma_u(z) = \text{standard deviation of the instantaneous wind speed at the height } z \text{ above the sea surface, in m/s (ft/s)}
\]

\[
\nu = \text{zero up-crossing frequency of wind speed, in Hz}
\]

\[
\nu = [0.007 + 0.213(T_u/\tau)^{0.654}]/T_u
\]

\[
T_u = \text{longitudinal integral length time scale, in seconds}
\]

\[
T_u = 3.12 z^{0.2}
\]

The gust factor associated with one hour reference time period is relate to the peak factor through:

\[
G(z, \tau, T_0 = 3600s) = 1 - g(z, \tau, T_0 = 3600s)\sigma_u(z)
\]

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 \( z = z_{ref} = 10 \text{ m (33 ft) above the sea surface}, \)

\[
V_{10min}(z_{ref}) = 1.03 V_{1hr}(z_{ref})
\]

For a reference time period \( T_0 (> \tau) \text{ smaller than 3600 s, the gust factor can be calculated by:} \)

\[
G(z, \tau, T_0) = [0.2193\ln(\log_{10}T_0) - 0.7242] \times G(z, \tau, T_0 = 3600s)
\]
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FIGURE 1  Flowchart of Time Domain Fatigue Analysis Method .......... 133
1 General

Spectral fatigue analysis method has been commonly 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/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 October 2015)
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.

\[
(H_s)_{eq} = \left[ \frac{\sum_{i=1}^{N} p_i (H_s)_i^m}{\sum_{i=1}^{N} p_i} \right]^{1/m}
\]

\[
(T_p)_{eq} = \left[ \frac{\sum_{i=1}^{N} p_i (T_p)_i^m}{\sum_{i=1}^{N} p_i} \right]^{1/m}
\]

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

1 Abbreviations

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

3 References (15 January 2013)

1. ABS Guide for Buckling and Ultimate Strength Assessment for Offshore Structures
2. ABS Guide for Building and Classing Mobile Offshore Units
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4. ABS Guide for Nondestructive Inspection of Hull Welds
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11. ACI 301, “Specifications for Structural Concrete”
12. ACI 318, “Building Code Requirements for Reinforced Concrete”
15. ACI 359, “Code for Concrete Reactor Vessels and Containments”
17. ACI 213R, “Guide for Structural Lightweight Aggregate Concrete”
18. AISC “Manual of Steel Construction”
<table>
<thead>
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<tr>
<td>23.</td>
<td>ASTM C172, “Standard Practice for Sampling Freshly Mixed Concrete”</td>
</tr>
<tr>
<td>24.</td>
<td>ASTM C31, “Standard Practice for Making and Curing Concrete Test Specimens in the Field”</td>
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<tr>
<td>28.</td>
<td>AWS, “Structural Welding Code – Steel, D1.1”</td>
</tr>
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<td>33.</td>
<td>ISO 2533, “Standard Atmosphere”</td>
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<td>34.</td>
<td>ISO 19906, “Petroleum and natural gas industries -- Arctic offshore structures”</td>
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<td>NACE SP0108, “Standard Practice: Corrosion Control of Offshore Structures by Protective Coatings”</td>
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<td>36.</td>
<td>NACE SP0176, “Standard Practice: Control of Submerged Areas of Permanently Installed Steel Offshore Structures Associated with Petroleum Production”</td>
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