GUIDE FOR BUILDING AND CLASSING

OFFSHORE WIND TURBINE INSTALLATIONS

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Foreword

This Guide provides criteria for the design, construction, installation and survey of permanently sited support structures for offshore wind turbines.

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

The design requirements specified in this Guide are not intended for application to floating offshore wind turbine installations.

GUIDE FOR BUILDING AND CLASSING

OFFSHORE WIND TURBINE INSTALLATIONS

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CHAPTER 1 Classification and Surveys

SECTION 1 Scope and Conditions of Classification

1 Classification

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

Additional requirements specific to Offshore Wind Turbine Installations are contained in the following Sections.

3 Classification Symbols and Notations

The following class notations are designated for Offshore Wind Turbine Installations.

3.1 Installations Built under ABS Survey

Installations built and constructed to the satisfaction of ABS Surveyor and to the requirements of this Guide or to their equivalent, where approved by ABS for the intended services and conditions, are to be classed and distinguished in the ABS *Record* by the following symbol:

✠ A1 Offshore Wind Turbine Installation

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

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 are classed with the symbol:

A1 Offshore Wind Turbine Installation

3.5 Additional Notations

3.5.1 Design Environmental Conditions

The maximum design environmental condition is identified in the ABS *Record* by the added notation **S (years)**. For example, **S (100)** indicates that the maximum site-specific design environmental conditions have a return period of 100 years.

3.5.2 Fatigue Life

The design fatigue life is distinguished in the ABS *Record* by the added notation **FL (years)**. For example, **FL (20)** indicates the minimum design fatigue life assessed is 20 years. The **(years)** refers to the fatigue life equal to 20 years or more (in 5-year increments), as specified by the Owner. Only one design fatigue life is published for the entire structural system. Where differing design fatigue life values are intended for different structural elements within the installation, the **(years)** refers to the least of the varying target lives. The "design fatigue life" refers to the target value set by the Owner, not the value calculated in the analysis.

5 Rules for Classification

5.1 Application

The requirements in this Guide are applicable to the Support Structure of an Offshore Wind Turbine, as defined in 1-4/3.53.

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 2 contains a list of such references. Unless otherwise noted, the current issue of the reference is to be used.

7 Design Documentation to be Submitted

The design documentation to be submitted is to describe the data, tools, procedures and methodologies of design and analysis which were employed to establish the design of the Support Structure of an Offshore Wind Turbine. The intended design service 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 the uniqueness of the structure or the lack of experience with conditions in the area where the structure is to be located.

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

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

The report is also to contain the information of wind turbines that is used as the input or as the basis of the input for the load calculation and structural assessment for the Support Structure of an Offshore Wind Turbine.

7.1.2 Environmental Considerations

Reports on environmental considerations are to describe all environmental phenomena appropriate to the areas of construction, transportation, installation and maintenance. The types of environmental phenomena to be accounted for, as appropriate to the type and location of the structure, 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 the environmental parameters is to be based on appropriate original data or, when permitted, data from analogous areas. Demonstrably valid statistical models to extrapolate to long-term values are to be employed. Any calculations required to establish the pertinent environmental parameters are to be submitted.

The report on environmental considerations is also to contain the calculations which quantify the effects or loadings on the structure where these 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.

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 as well as structural dynamic properties and foundation designs. Calculations are in general to be compliance with 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

Calculations of natural periods of the Support Structure, including the tower, substructure and foundation, 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 of an offshore wind turbine 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 be required for the stresses in localized areas and structural joints, the dynamic response of the structure, and fatigue life of critical members and joints.

For pile supported structures, calculations for the stresses in piles and the load capacity of the connection between the structure and pile are to be submitted. Similarly, for gravity structures, 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

As applicable, calculations are to be submitted in compliance with Chapter 6. When accounting for the structural responses described above in 1-1/7.3.3 and those resulting from considerations of marine operations in Chapter 6, calculations are to demonstrate the adequacy of the structural elements, members or local structure. In addition, the calculations are to demonstrate, as applicable, that the deflections resulting from the 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

Plans or specifications depicting or describing the arrangements and details of the major items of the 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 the submitted data and ABS will adhere to as far as reasonably possible, is to reflect the construction schedule and the complexity of Offshore Wind Turbine Installations.

Generally, plans and specifications are to be submitted electronically to ABS. However, hard copies will also be accepted. 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 and weights of turbine components (e.g., blade, hub, shaft, 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. Pile plans indicating arrangements, nominal sizes, thicknesses and penetration.

v) Welding details and procedures, and schedule of nondestructive testing.

vi) Corrosion control systems.

vii) Various information in support of novel features utilized in the Support Structure 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

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 Rotor-Nacelle Assemblies (RNAs) to be installed.

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

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

iv) Plans showing the general arrangement of Offshore Wind Turbine Installations.

v) Description of the safety and protective systems provided.

vi) Description of the modes of operation.

vii) Listing of governmental authorities having cognizance over the installation.

viii) Listing of any novel features.

ix) Brief description of any monitoring proposed for use on the installation.

x) Description of transportation, installation and maintenance procedures.

9 Operating Manual

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

CHAPTER 1 Classification and Surveys

SECTION 2 Surveys During Construction and Installation

1 Overview

1.1 Scope

This Section pertains to surveys during the construction and installation of the Support Structure of an offshore wind turbine. The requirements of 1-2/1 are to apply to all 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 Rotor-Nacelle Assembly (RNA).

1.3 Quality Control Program

A quality control program compatible with the type and size of the planned Support Structure of an offshore wind turbine is to be developed and submitted to ABS for review. ABS will review, approve and, as necessary, request modification of this program. The Fabricator is to work with the attending Surveyor to establish the required hold points on the quality control program to form the basis for all future surveys at the fabrication yard. As a minimum, the items enumerated in the various applicable Subsections below are to be covered by the quality control program. Surveyors will be assigned to monitor the fabrication of items within the scope of classification, and to verify that competent personnel are carrying out all tests and inspections specified in the quality control program. It is to be noted that the monitoring provided by ABS is a supplement to and not a replacement for inspections to be carried out by the Fabricator or Operator.

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 and 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. 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 are to work on the Support Structure of an offshore wind turbine are to be qualified in accordance with the welder qualification tests specified in a recognized code or, as applicable, the ABS *Rules for Materials and Welding (Part 2)* to the satisfaction of the attending Surveyor. Certificates of qualification are to be prepared to record evidence of the qualification of each welder qualified by an approved standard/code, and such certificates are to be available for the use of 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 being met. Particular attention is to be paid to the out-of-roundness of members for which buckling is an anticipated mode of failure. Structural alignment and fit-up prior to welding are to be monitored to verify consistent production of quality welds.

3.15 Corrosion Control Systems

The details of any corrosion control systems employed for the structure are to be submitted for review. Installation and testing of the corrosion control systems are to be carried out to the satisfaction of the attending Surveyor in accordance with the approved plans.

3.17 Tightness and Hydrostatic Testing Procedures

Compartments, which are designed to be permanently watertight or to be maintained watertight during installation, are to be tested by a procedure approved by the attending Surveyor. The testing is also to be witnessed by the attending Surveyor.

3.19 Nondestructive Testing

A system of nondestructive testing is to be included in the fabrication specification of the structures. The minimum extent of nondestructive testing is to be in accordance with this Guide or recognized design Code. All nondestructive testing records are to be reviewed and approved by the attending Surveyor. Additional nondestructive testing may be requested by the attending Surveyor if the quality of fabrication is not in accordance with industry standards.

3.21 Installation of 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 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 structural 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 classing the structure.

After fabrication and installation, these records are to be retained by the Operator 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 who will check conformity with the properties specified in the approved design.

As applicable, at the construction site, the Surveyor is to be satisfied that proper consideration is being given to the support of the structure during construction, the storage of cement and prestressing tendons in weathertight areas, the storage of admixtures and epoxies to manufacturer's specifications, and the storage of aggregates to limit segregation, contamination by deleterious substances and moisture variations within the stock pile.

Forms and shores supporting the forms are to be inspected to verify that they are adequate in number and type, and that they are located in accordance with the approved plans. The dimensions and alignment of the forms are to be verified by the attending Surveyor. The measurements are to be within the allowable finished dimensional tolerances specified in the approved design.

Reinforcing steel, prestressing tendons, post-tensioning ducts, anchorages and any included steel are to be checked, as appropriate to the planned structure, for size, bending, spacing, location, firmness of installation, surface condition, vent locations, proper duct coupling, and duct capping.

5.5 Inspection of Batching, Mixing, and Placing Concrete

The production and placing of the concrete are to employ procedures which will provide a well mixed and well compacted concrete. Such procedures are also to limit segregation, loss of material, contamination, and premature initial set during all operations.

Mix components of each batch of concrete are to be measured by a method specified in the quality control program. The designer is to specify the allowable variation of mix component proportions, and the Fabricator is to record the actual proportions of each batch.

Testing during the production of concrete is to be carried out following the procedures specified in the quality control program. As a minimum, the following concrete qualities are to be measured by the Fabricator.

i) Consistency

ii) Air content

iii) Density or Specific Gravity

iv) Strength

Field testing of aggregate gradation, cleanliness, moisture content, and unit weight is to be performed by the Fabricator following standards and schedules specified in the quality control program. The frequency of testing is to be determined with the 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 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 Support Structure of an offshore wind turbine is in service.

5.13 Inspection of Finished Concrete

The surface of the hardened concrete is to be completely 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 which are 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 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 Operator.

7 Rotor-Nacelle Assemblies (RNAs)

The Surveyor is to witness the installation of at least one RNA per each type. Where there are more than 10 RNAs of the same type, at least one RNA installation per every 10 turbines of the same type is 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 Support Structures. Deviations from approved design documents and plans or any incidents such as damage or overstress to the Support Structure 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.

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 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 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.

1.3 Notification and Availability for Survey

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

The Surveyor is to undertake all surveys on classed Offshore Wind Turbine Installations upon request, with adequate notification, from the Owners or their representatives and is to report thereon to ABS. Should the Surveyor finds 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 1-1-4/9 of the ABS *Rules for Conditions of Classification – Offshore Units and Structures (Part 1)*.

1.5 Annual Surveys

Annual Surveys of the 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 Operator, 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 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 Operator, 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 Operator, 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 Support Structure 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 Support Structure 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 Operator 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 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 and the issuance and use of any such reports or certificates are to be, in all respects, governed by 1-1/1.

1.17 Survey of a Self-Elevating Unit to be Used as the Support Structure

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*. Surveys are to include Annual and Special Periodical Surveys with an underwater exam in lieu of drydocking of the above mudline sections of the legs, mats, spud cans and platform twice in each five year Special Periodical Survey period and in accordance with applicable sections of the ABS *Guide for Building and Classing Mobile Offshore Units*. Spud cans and mats located below the mud line are to be considered inaccessible. Fatigue, structural and corrosion analyses are to be provided to justify the integrity of these inaccessible areas for the design life of the installation.

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 to determine the actual condition of the Support Structure. 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 Support Structure.

The need for more frequent future periodical surveys is to be determined based on the calculated remaining fatigue life described in Part 4, Section 1 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. Survey requirements described in Chapter 1, Section 2 and 1-3/1.19, wherever applicable, are to be complied with in addition to an 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 all 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 Operator 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 Part 4, Section 1 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 an 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 Support Structure. 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 Operator 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 Support Structure 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 Part 4, Section 1 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 an 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 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 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 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 Support Structure, but will not contribute to the strength of the Support Structure. 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 Operator decides to leave the marine growth greater than what is allowed in the approved design, the Operator is to justify that the higher hydrodynamic loading due to the additional marine growth will not affect the structural integrity of the structure. The Operator 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 Operator is prepared to maintain.

15 Maintenance of Rotor-Nacelle Assembly

The Operator 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.

iii) All repair, modification and replacement have been carried out in accordance with the manufacturers' recommendations.

17 Statutory Certification

When ABS is authorized to perform certification on behalf of a governmental authority, or when requested by the Operator, requirements as specified by the governmental authority or Operator are to be certified accordingly and the reports and certificates in accordance with those requirements are to be issued as appropriate.

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

1.7 Compliant Tower

A structure that consists of a slender tower supported at the sea floor by an installed foundation or by a large spud can and may also be partially supported by buoyancy aids. Guy lines may or may not be used for lateral restraints.

3 Terminology

3.1 Application of Structural Members

The application of structural members in the Support Structure of an offshore wind turbine is to be in accordance with the categories listed in this paragraph.

3.1.1 Special Application Structure

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

3.1.2 Primary Application Structure

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

3.1.3 Secondary Application Structure

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

3.3 Consultant

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

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

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

3.7 Cut-Out Wind Speed (V_{out})

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

3.9 Emergency Shutdown

Rapid shutdown of the wind turbine triggered by a protection function or by manual intervention

3.11 Fabricator

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

3.13 Gust

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

3.15 Hub Height

Height of the center of the swept area of the wind turbine rotor above the Mean Sea Level.

3.17 Idling

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

3.19 Mean Sea Level (Mean Still Water Level)

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

3.21 Mean Wind Speed

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

3.23 Normal Shutdown

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

3.25 Offshore Wind Farm

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

3.27 Offshore Wind Turbine

An Offshore Wind Turbine consists of a Rotor-Nacelle Assembly and its Support Structure, as defined in 1-4/3.45 and 1-4/3.53, respectively.

3.29 Offshore Wind Turbine Installation

An Offshore Wind Turbine Installation consists of the Support Structure, which is defined in 1-4/3.53, for an Offshore Wind Turbine. The site-specific data for an Offshore Wind Turbine Installation employed by the designer and submitted for review by ABS will form a part of its classification/certification.

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

3.31 Omni-directional (Wind, Waves or Currents)

Acting in all directions

3.33 Operator

An operator is any person or organization empowered to conduct operations on behalf of the Owners of Offshore Wind Turbines.

3.35 Owner

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

3.37 Parked

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

3.39 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.

3.41 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

3.43 Return 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.

3.45 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.

3.47 Splash Zone

Part of a Support Structure containing the areas above and below the Still Water Level and regularly subject to wetting due to wave and tide action

3.49 Standstill

Condition of a wind turbine that is not rotating

3.51 Still Water Level (SWL)

Still Water Level (SWL) is taken as the sum of the highest astronomical level and the positive storm surge excluding variations due to waves.

3.53 Support Structure

A Support Structure of offshore wind turbine is a site dependent offshore structure, supported by or attached to the sea floor. The design of the Support Structure is based on foundation and long term environmental conditions at a particular installation site where it is intended to remain. The sea floor attachment afforded to the Support Structure may be obtained by pilings, direct bearing, or other types of foundation.

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

3.53.1 Tower

Structure component which extends upward from somewhere above the Mean Sea Level and connects the Substructure to the Rotor-Nacelle assembly

3.53.2 Substructure

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

3.53.3 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

3.55 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.

3.57 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

3.59 Uni-directional (Wind, Waves or Currents)

Acting in a single directions

3.61 Water Depth

Vertical distance between the sea floor and Still Water Level

3.63 Wind Profile (Wind Shear Law)

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

3.65 Yawing

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

3.67 Yaw Misalignment

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

CHAPTER 2 Materials and Welding

CONTENTS

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 a 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 Appendix A 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 subject 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

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

Some of the typical ASTM and API steels belonging to the groups of 2-1/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

Group	Specification & Grade	Yield Strength		Tensile Strength	
		ksi	MPa	ksi	MPa
I	ASTM A36	36	250	58-80	400-550
	ASTM A131 Grade A (ABS Grade A)	34	235	58-75	400-515
	ASTM A285 Grade C	30	205	55-75	380-515
	ASTM A131 Grades B, D (ABS Grades B, D)	34	235	58-75	400-515
	ASTM A516 Grade 65	35	240	65-85	450-585
	ASTM A573 Grade 65	35	240	65-77	450-530
	ASTM A709 Grade 36T2	36	250	58-80	400-550
	ASTM A131 Grade E (ABS Grade E) (ABS Grade E)	34	235	58-75	400-515
II	ASTM A572 Grade 42	42	290	60 min.	415 min.
	ASTM A572 Grade 50	50	345	65 min.	450 min.
	ASTM A588 (to 2 in. thick)	50	345	70 min.	485 min.
	ASTM A709 Grades 50T2, 50T3	50	345	65 min.	450 min.
	ASTM A131 Grade AH32 (ABS Grade AH32)	46	315	68-85	470-585
	ASTM A131 Grade AH36 (ABS Grade AH36)	51	350	71-90	490-620
	API Spec 2H-Grade 42	42	290	62-80	425-550
	API Spec 2H-Grade 50 (to $2^1/_2$ in. thick)	50	345	70-90	485-620
	(over $2^1/_2$ in. thick)	47	325	70-90	485-620
	API Spec 2W-Grade 42 (to 1 in. thick)	42-67	290-460	62 min.	425 min.
	(over 1 in. thick)	42-62	290-430	62 min.	425 min.
	Grade 50 (to 1 in. thick)	50-75	345-515	65 min.	450 min.
	(over 1 in. thick)	50-70	345-485	65 min.	450 min.
	Grade 50T (to 1 in. thick)	50-80	345-550	70 min.	485 min.
	(over 1 in. thick)	50-75	345-515	70 min.	485 min.
	API Spec 2Y-Grade 42 (to 1 in. thick)	42-67	290-460	62 min.	425 min.
	(over 1 in. thick)	42-62	290-430	62 min.	425 min.
	Grade 50 (to 1 in. thick)	50-75	345-515	65 min.	450 min.
	(over 1 in. thick)	50-70	345-485	65 min.	450 min.
	Grade 50T (to 1 in. thick)	50-80	345-550	70 min.	485 min.
	(over 1 in. thick)	50-75	345-515	70 min.	485 min.
	ASTM A131 Grades DH32, EH32 (ABS Grades DH32, EH32	46	315	68-85	470-585
	Grades DH36, EH36 (ABS Grades DH36, EH36)	51	350	71-90	490-620
	ASTM A537 Class 1 (to $2^1/_2$ in. thick)	50	345	70-90	485-620
	ASTM A633 Grade A	42	290	63-83	435-570
	Grades C, D	50	345	70-90	480-620
	ASTM A678 (80) Grade A	50	345	70-90	485-620
	ASTM A992	50-65	345-450	65 min.	450 min.
III	ASTM A537 Class 2	60	415	80-100	550-690
	ASTM A633 Grade E	60	415	80-100	550-690
	ASTM A678 (80) Grade B	60	415	80-100	550-690
	API Spec 2W-Grade 60 (to 1 in. thick)	60-90	415-620	75 min.	515 min.
	(over 1 in. thick)	60-85	415-585	75 min.	515 min.
	API Spec 2Y-Grade 60 (to 1 in. thick)	60-90	415-620	75 min.	515 min.
	(over 1 in. thick)	60-85	415-585	75 min.	515 min.
	ASTM A710-Grade A Class 3 (to 2 in. thick)	75	515	85 min.	585 min.
	(2 in. to 4 in. thick)	65	450	75 min.	515 min.
	(over 4 in. thick)	60	415	70 min	485 min.

TABLE 3
Structural Steel Pipes

Group	*Specification & Grade*	*Yield Strength*		*Tensile Strength*	
		ksi	*MPa*	*ksi*	*MPa*
I	API 5L-Grade B	35	240	60 min	415 min.
	ASTM A53 Grade B	35	240	60 min.	415 min.
	ASTM A135 Grade B	35	240	60 min.	415 min.
	ASTM A139 Grade B	35	240	60 min.	415 min.
	ASTM A381 Grade Y35	35	240	60 min.	415 min.
	ASTM A500 Grade A	33-39	230-270	45 min.	310 min.
	ASTM A501	36	250	58 min.	400 min.
	ASTM A106 Grade B	35	240	60 min.	415 min.
	ASTM A524 (strength varies with thickness)	30-35	205-240	55-85	380-585
II	API 5L95 Grade X42 (2% max. cold expansion)	42	290	60 min.	415 min.
	API 5L95 Grade X52 (2% max. cold expansion)	52	360	66 min.	455 min.
	ASTM A500 Grade B	42-46	290-320	58 min.	400 min.
	ASTM A618 Grade Ia, Ib & II (to $^3/_4$ in. thick)	50	345	70 min.	485 min.
	API 5L95 Grade X52 (with SR5, SR6, or SR8)	52	360	66 min.	455 min.

1.5.3 Toughness Characteristics

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

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

ii) Demonstration that a steel manufactured by a particular producer using a specific manufacturing process has minimum toughness levels representative of those listed herein.

iii) Charpy impact testing in accordance with 2-1/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 5-1/5.9), the service temperature is defined as 0°C (32°F). A higher service temperature may be used if adequate supporting data can be presented relative to the lowest average daily water temperature applicable to the depths involved.

1.7.2 Material within or above the Splash Zone

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

1.7.3 Special Conditions

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

TABLE 4
Charpy Toughness Specification for Steels

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

Notes

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

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

Group	*Test Temperature*	*Minimum Service Temperature (As determined by 2-1/1.7)*
I, II	30°C (54°F) below Minimum Service Temperature	
III	–40°C (–40°F)	0°C (32°F)
	–50°C (–58°F)	–10°C (14°F)
	–50°C (–58°F)	–20°C (–4°F)
	–60°C (–76°F)	–30°C (–22°F)

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

Group	*Test Temperature*	*Minimum Service Temperature (As determined by 2-1/1.7)*
I, II	10°C (18°F) below Minimum Service Temperature	
III	–40°C (–40°F)	0°C (32°F)
	–40°C (–40°F)	–10°C (14°F)
	–40°C (–40°F)	–20°C (–4°F)
	–50°C (–58°F)	–30°C (–22°F)

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

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

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

2 Impact tests are not necessary for section sizes below 6 mm (0.25 in.) in thickness.

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: $^{2}/_{3}$ 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 5-1/5.9) to the considerations indicated.

i) *Submerged zone:* chemical deterioration of the concrete, corrosion of the reinforcement and hardware, and abrasion of the concrete.

ii) *Splash zone:* freeze-thaw durability, corrosion of the reinforcement and hardware, chemical deterioration of the concrete, and fire hazards.

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

iv) *Atmospheric zone:* freeze-thaw durability, corrosion of reinforcement and hardware, and fire hazards.

3.3 Cement

3.3.1 Type

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

3.3.2 Tricalcium Aluminate

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

3.3.3 Oil Storage

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

3.5 Water

3.5.1 Cleanliness

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

3.5.2 Non-potable Water

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

3.7 Chloride or Sulfide Content

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

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

3.9 Aggregates

3.9.1 General

Aggregates are to conform to the requirements of ASTM C33 or equivalent. Other aggregates may be used if there is supporting evidence that they produce concrete of satisfactory quality. When specially approved, lightweight aggregates similar to 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.

5.9.3 Size

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

3.11 Admixtures

3.11.1 General

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

3.11.2 Recognized Standards

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

3.11.3 Pozzolan Content

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

3.13 Steel Reinforcement

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

3.13.1 Reinforcement for Non-Prestressed Concrete

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

i) Deformed reinforcing bars and plain bars: ASTM A615

ii) Bar and rod mats: ASTM A184

iii) Plain wire for spiral reinforcement: ASTM A82, ASTM A704

iv) Welded plain wire fabric: ASTM A185

v) Deformed wire: ASTM A496

vi) Welded deformed wire fabric: ASTM A497

3.13.2 Welded Reinforcement

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

3.13.3 Steel Reinforcement for Prestressed Concrete

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

i) Seven-wire strand: ASTM A416

ii) Wire: ASTM A421

3.13.4 Other Materials

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

3.15 Concrete

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

3.17 Water-Cement Ratios

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

TABLE 5
Water-Cement Ratios and Compressive Strengths

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

* Depending upon severity of exposure

3.19 Other Durability Requirements

3.19.1 Cement Content

Minimum cement content is to verify an adequate amount of paste for reinforcement protection and generally be not less than 355 kg/m^3 (600 lb/yd^3).

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 $t/2$ or 12.5 mm ($^1/_2$ in.), whichever is less, where t is the thickness of the thinner member being welded. Where sections to be butt welded differ in thickness and have an offset on any side of more than 3 mm ($^1/_8$ in.), a suitable transition taper is to be provided. In general, the transition taper length is to be not less than three times the offset. The transition may be formed by tapering the thicker member or by specifying a weld joint design which will provide the required transition.

5.3 Alignment

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

5.5 Cleanliness

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

5.7 Tack Welds

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

5.9 Run-on and Run-off Tabs

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

7 Production Welding

7.1 Environment

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

7.3 Sequence

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

7.5 Preheat and Postweld Heat Treatment

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

7.7 Low-hydrogen Electrodes or Welding Processes

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

7.9 Back Gouging

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

7.11 Peening

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

7.13 Fairing and Flame Shrinking

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

7.15 Weld Soundness and Surface Appearance

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

7.17 Inspection of Welds

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

7.19 Extent of Inspection of Welds

7.19.1 General

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 hydro static 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 subject 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 ($^1/_{16}$ in.) may be permitted provided that the penetration at the root is at least 1.5 mm ($^1/_{16}$ 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.

CHAPTER 3 Environmental Conditions

CONTENTS

CHAPTER 3 Environmental Conditions

SECTION 1 Overview

1 General

The environmental conditions to which an offshore wind turbine installation is expected to be exposed during its life are to be described 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.

The environmental phenomena that influence the transport, installation, maintenance and operation of an offshore wind turbine installation are to be described in terms of characteristic parameters relevant to the evaluation of the installation. 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 of data noted.

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 the environmental phenomena under consideration, as evidenced by relevant statistical tests, confidence limits and other measures of statistical significance. Hindcasting methods and models are to be fully documented if they are used to derive the environmental data.

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

3 Environmental Factors to be Considered

In general, the design of an offshore wind turbine installation 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 the specific installation site.

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

3.1 Wind Speed and Turbulence

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

In this Guide, the mean wind speed, denoted as V_{hub}, at turbine hub height with 10-minute averaging duration is employed to define the turbulent wind conditions in the Design Load Cases (DLCs) in 4-2/3. The steady wind conditions as referred in the DLCs in 4-2/3 are defined using the mean wind speed with one minute or 3-second averaging duration.

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 $t = 3600/V_f$, where V_f is the fastest mile of wind at a reference height of 10 m (32.8 ft), in miles per hour.

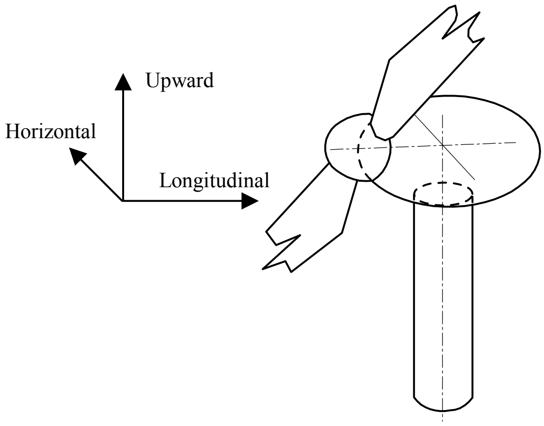
The turbulence of wind over a duration of 10 minutes is generally considered stationary and can be modeled by power spectral density functions and coherence functions. The turbulence model is to include the effects of varying wind speed, shears and directions and allow rotational sampling through varying 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

FIGURE 1
Vector Components of Turbulent Wind Velocity



3.3 Wind Profile

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 turbine hub height, in m/s (ft/s)

α = power law exponent, values of which are given in 3-2/9 and 3-2/11

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

z_{hub} = hub height above the SWL, in m (ft)

For strong wind conditions, such as a hurricane wind in the Gulf of Mexico with return period in excess of 10 years, the mean wind speed profile may be represented by the following logarithmic wind shear law, which is expressed using the 1-hour mean wind speed at 10 m (32.8 ft) above the SWL.

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

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)

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

t_0 = reference averaging time period (1 hour), in seconds

= 3600 s

C = $0.0573\sqrt{1 + 0.15V_0/\phi}$

V_0 = 1-hour mean wind speed at 10 m (32.8 ft) above the SWL, in m/s (ft/s)

$I_u(z)$ = turbulence intensity at height z

$$= 0.06[1 + 0.043V_0/\phi]\left(\frac{z}{10\phi}\right)^{-0.22}$$

ϕ = unit conversion factor

= 1 when using SI units (m, m/s)

= 3.28 when using US Customary units (ft, ft/s)

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

3.5 Wind Spectrum and Spatial Coherence

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 hurricane-prone offshore sites, such as the Gulf of Mexico and North Sea, API RP 2A-WSD (2007) recommends using the NPD wind spectrum (also known as Frøya model) in conjunction with the two-point coherence function and the logarithmic wind shear law to model hurricane wind conditions. 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 document. 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

A wind condition for offshore wind turbine design is represented by a constant mean flow combined with either a varying deterministic gust profile or turbulence. The design wind conditions are further categorized into the normal wind conditions, which occur more frequently than once per year during normal operation of an offshore wind turbine, and the extreme wind conditions representing rare wind conditions with 1-year and 100-year return period. The extreme wind condition with less than 100-year return period may be applied in accordance with 4-2/3.

The normal and extreme wind conditions for the design of the Support Structure of an offshore wind turbine are specified in 3-2/9 and 3-2/11. The Design Load Case (DLC) descriptions in 4-2/Table 1 indicate which wind condition is to be applied in each DLC.

9 Normal Wind Conditions

9.1 Normal Wind Profile Model (NWP)

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 $\alpha = 0.14$.

9.3 Normal Turbulence Model (NTM)

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

The value of the turbulence standard deviation is to be determined using appropriate statistical techniques applied to measured and preferably de-trended data. Where the site assessment is not available, the recommended approach provided in Section 12.3 of IEC 61400-3 (2009) may be used to estimate the standard deviation where applicable.

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

The extreme wind conditions are represented by extreme wind shear events, peak wind speeds due to storms, and rapid changes in wind speed and direction.

11.1 Extreme Wind Speed Model (EWM)

The EWM is defined as either a steady or a turbulent wind model with a specified return period. Both the 100-year and 1-year return extreme wind conditions are to be considered in the Design Load Cases specified in 4-2/Table 1.

When site data are not available, the wind profile of 10-minute mean wind speeds for the turbulent extreme wind speed model with return periods of 100 years and 1 year, respectively, is to be represented by the power law model as follows:

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

where

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

= $V_{10\text{min},100\text{-yr}}$ for 100-year return extreme wind condition

= $V_{10\text{min},1\text{-yr}}$ for 1-year return extreme wind condition

$V_{10\text{min},100\text{-yr}}$ = 10-minute mean wind speed at hub height with a return period of 100 years, in m/s (ft/s)

$V_{10\text{min},1\text{-yr}}$ = 10-minute mean wind speed at hub height with a return period of one year, in m/s (ft/s)

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

z_{hub} = hub height above the SWL, in m (ft)

The standard deviation of longitudinal turbulent wind speed of extreme wind condition, σ_1, 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 with 100-year return period.

For the steady extreme wind model, the 100-year and 1-year return wind speeds are to be assumed as a function of height, z, above the SWL using the following equations:

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

where

$V(z_{hub})$ = steady extreme wind speed at hub height, in m/s (ft/s)

= $V_{3sec,100\text{-}yr}$ for 100-year return extreme wind condition

= $V_{3sec,1\text{-}yr}$ for 1-year return extreme wind condition

$V_{3sec,100\text{-}yr}$ = 3-second mean wind speed at hub height with a return period of 100 years, in m/s (ft/s)

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

In the steady extreme wind model, constant yaw misalignment in the range of ±15° is to be applied to account for short-term deviations from the mean wind direction.

11.3 Reduced Extreme Wind Speed Model (RWM)

The 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 1-year or 100-year return periods are to be assessed based on site-specific environmental data.

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

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

where

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

= $V_{10min,100\text{-}yr}$ for 100-year return extreme wind condition

= $V_{10min,1\text{-}yr}$ for 1-year return extreme wind condition

$V_{10min,100\text{-}yr}$ = 1-minute mean wind speed at hub height with a return period of 100 years, in m/s (ft/s)

$V_{10min,1\text{-}yr}$ = 1-minute mean wind speed at hub height with a return period of one year, in m/s (ft/s)

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

z_{hub} = hub height above the SWL, in m (ft)

Constant yaw misalignment in the range of ±15° is to be applied to account for short-term deviations from the mean wind direction.

11.5 Extreme Operating Gust (EOG)

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

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

where

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

Λ_1 = longitudinal turbulence length scale, in m (ft)

= $0.7z$ when $z \leq 60$ m (196.8 ft)

= 42 m (137.8 ft) when $z \geq 60$ 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:

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

where

$V(z)$ = normal wind profile defined in 3-3/9.1, in m/s (ft/s)

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

T = 10.5 s

11.7 Extreme Turbulence Model (ETM)

The ETM is to be represented by the normal wind profile (NWP) model specified in 3-2/9.1 and the turbulence whose standard deviation of longitudinal component is given by:

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

where

c = 2 m/s (6.56 ft/s)

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

V_{ave} = site-specific annual mean wind speed at hub height, in m/s (ft/s)

I_{ref} = expected value of turbulence intensity at hub height when V_{hub} =15 m/s (49.2 ft/s)

11.9 Extreme Direction Change (EDC)

The extreme direction change magnitude, θ_e, is to be calculated by:

$$\theta_e = \pm 4\arctan\left(\frac{\sigma_{NTM}}{V_{hub}\left(1 + 0.1D/\Lambda_1\right)}\right) \qquad -180° \le \theta_e \le 180°$$

where

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

Λ_1 = longitudinal turbulence length scale, defined in 3-2/11.5, in m (ft)

D = rotor diameter, in m (ft)

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

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

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

11.11 Extreme Coherent Gust with Direction Change (ECD)

The extreme coherent gust with direction change is to have a magnitude of:

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

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

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

where

$V(z)$ = normal wind profile defined in 3-2/9.1, in m/s (ft/s)

T = rise time of gust wind

= 10 s

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

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

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

where

θ_{cg} = magnitude of direction change, in degree

$$= \begin{cases} 180° & V_{hub} \le 4\text{ m/s } (13.12\text{ ft/s}) \\ 720°(V_{hub} / \phi)^{-1} & V_{hub} > 4\text{ m/s } (13.12\text{ ft/s}) \end{cases}$$

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

T = rise time of gust wind

= 10 s

ϕ = unit conversion factor

= 1 when using SI units (m, m/s)

= 3.28 when using US Customary units (ft, ft/s)

11.13 Extreme Wind Shear (EWS)

The extreme wind shear 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:

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

The time history of transient horizontal shear is given by:

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

where

σ_{NTM}	=	longitudinal turbulence standard deviation defined in 3-2/9.3, in m/s (ft/s)	
Λ_1	=	longitudinal turbulence length scale, defined in 3-2/11.5, in m (ft)	
z	=	height above the SWL, in m (ft)	
x_{hub}	=	hub height above the SWL, in m (ft)	
y	=	horizontal distance from hub in the cross wind direction, in m (ft)	
D	=	rotor diameter, in m (ft)	
α	=	0.14	
β	=	6.4	
ϕ	=	unit conversion factor	
	=	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

The development of wave data is to reflect conditions at the installation site and the type of 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 are to be used where such exist. Hindcasting techniques that adequately account for 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 they 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) 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 for use in either deterministic or probabilistic methods of analysis as appropriate. Waves that cause the most unfavorable effects on the overall structure may 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 return period chosen for the severe or extreme wave conditions, as defined in 3-3/5 and 3-3/7, is generally not to be less than 100 years. A reduction to this requirement of return period is to be in accordance with 4-2/3.

The Design Load Cases (DLCs) listed in 4-2/Table 1 are specified for normal, severe or extreme wave conditions as described below in this Section. The normal, severe and extreme wave conditions are characterized both in a stochastic fashion (i.e., sea state spectra) and deterministically.

3 Normal Wave Conditions

3.1 Normal Sea State (NSS)

The normal stochastic sea state is 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 is used in Chapter 4, Section 2 to define a number of Design Load Cases (DLCs) requiring either strength analysis or fatigue analysis. For fatigue calculations, the number and resolution of the normal sea states considered 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.

For strength calculations, the normal sea state can be characterized by the expected value of significant wave height, $H_{s,NSS}$, conditioned upon a given value of V_{hub} (i.e., $H_{s,NSS} = E[H_s | V_{hub}]$). A range of peak period, T_p, associated with each significant wave height is to be determined for load calculations. The resultant highest loads are to be used in the design of offshore wind turbine installation.

3.3 Normal Wave Height (NWH)

The normal deterministic design wave height, H_{NWH}, 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 normal deterministic wave height is to be determined for load calculations. The resultant highest loads are to be used in the design of offshore wind turbine installation. The effect of water depth at 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:

$$11.1\sqrt{\frac{H_{s,NSS}(V_{hub})}{g}} \le T \le 14.3\sqrt{\frac{H_{s,NSS}(V_{hub})}{g}}$$

where

$H_{s,NSS}(V_{hub})$ = expected value of the significant wave height of normal sea state conditioned upon a given value of V_{hub}, in m (ft), where V_{hub} is the 10-minute mean wind speed at hub height, in m/s (ft/s)

= normal wave height, H_{NWH}

g = acceleration of gravity, in m/s^2 (ft/s^2)

T = period associated with the NWH, 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 relationships may be used:

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

where

H_{NWH} = normal wave height, in m (ft)

T = period associated with the NWH, in seconds

d = water depth water depth measured from the SWL to the sea floor, in m (ft)

g = acceleration of gravity, in m/s^2 (ft/s^2)

c = 0.14 for a regular horizontal sea floor

5 Severe Wave Conditions

5.1 Severe Sea State (SSS)

The severe stochastic sea state condition is to be applied in combination with the normal wind conditions as specified in 3-2/9 for the strength analysis of the Support Structure of an offshore wind turbine during power production.

The severe sea state is represented by a significant wave height, $H_{s,SSS}$, 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 combination of $H_{s,SSS}$ and a given value of 10-minute mean wind speed, V_{hub}, at hub height has a return period of 100 years. A series of V_{hub} is to be selected within the range of mean wind speed corresponding to power production. As a conservative estimation, the 100-year return significant wave height independent of wind speed may be used to approximate $H_{s,SSS}$.

A range of peak period associated with each significant wave height is to be determined for load calculations. The resultant highest loads are to be used in the design of offshore wind turbine installation.

5.3 Severe Wave Height (SWH)

The severe deterministic design wave is to be applied in combination with the normal wind conditions as specified in 3-2/9 for the strength analysis of the Support Structure of an offshore wind turbine during power production. 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 combination of H_{SWH} and a given value of V_{hub} has a return period of 100 years. A series of V_{hub} is to be selected within the range of mean wind speed corresponding to power production. As a conservative estimation, the 100-year return extreme wave height independent of wind speed may be used to approximate H_{SWH}.

A range of wave period appropriate to each deterministic wave height is to be determined for load calculations. The resultant highest loads are to be used in the design of offshore wind turbine installation. The effect of water depth at 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:

$$11.1\sqrt{\frac{H_{s,SSS}(V_{hub})}{g}} \le T \le 14.3\sqrt{\frac{H_{s,SSS}(V_{hub})}{g}}$$

where

$H_{s,SSS}(V_{hub})$ = expected value of the significant wave height of severe sea state conditioned upon a given value of V_{hub}, in m (ft), where V_{hub} is the 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.

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

where

H_{SWH} = severe wave height, in m (ft)

T = period associated with the NWH, in seconds

d = water depth measured from the SWL to the sea floor, in m (ft)

g = acceleration of gravity, in m/s^2 (ft/s^2)

c = 0.14 for a regular horizontal sea floor

7 Extreme Wave Conditions

7.1 Extreme Sea State (ESS)

The extreme stochastic sea state model is to represent 100-year return or 1-year return wave conditions independent of wind speed. The significant wave height of the ESS model is denoted either as $H_{s,100\text{-yr}}$ or $H_{s,1\text{-yr}}$ for the extreme significant wave height with a return period of 100 years or one year, respectively. The values of $H_{s,100\text{-yr}}$ and $H_{s,1\text{-yr}}$ are to be determined from on-site measurements, hindcast data, or both for the installation site. Ranges of peak spectral periods appropriate to site-specific $H_{s,100\text{-yr}}$ and $H_{s,1\text{-yr}}$ respectively are to be determined for load calculations. The resultant highest loads for 100-year return conditions and one-year return conditions are to be used in the design of offshore wind turbine installation.

7.3 Extreme Wave Height (EWH)

The extreme deterministic design wave is to represent 100-year return or 1-year return wave conditions independent of wind speed. The extreme design wave height of the EWH model is denoted either as $H_{100\text{-yr}}$ or $H_{1\text{-yr}}$ for the extreme wave height with a return period of 100 years or one year, respectively.

The values of $H_{100\text{-yr}}$ and $H_{1\text{-yr}}$ are to be determined from on-site measurements, hindcast data, or both for the installation site. Ranges of wave periods appropriate to the site-specific $H_{100\text{-yr}}$ and $H_{1\text{-yr}}$ respectively are to be determined for load calculations.

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 to be considered may be estimated by the following equations for 100-year return conditions:

$$H_{100\text{-yr}} = 1.86\, H_{s,100\text{-yr}} \qquad 11.1\sqrt{\frac{H_{s,100-\text{yr}}}{g}} \le T \le 14.3\sqrt{\frac{H_{s,100-\text{yr}}}{g}}$$

and the following equations for 1-year return conditions:

$$H_{1\text{-yr}} = 1.86\, H_{s,1\text{-yr}} \qquad 11.1\sqrt{\frac{H_{s,1-\text{yr}}}{g}} \le T \le 14.3\sqrt{\frac{H_{s,1-\text{yr}}}{g}}$$

where

$H_{100\text{-yr}}$ = extreme wave height with a return period of 100 years, in m (ft)

$H_{s,100\text{-yr}}$ = significant wave height with a return period of 100 years for a 3-hour reference period, in m (ft)

$H_{1\text{-yr}}$ = extreme wave height with a return period of one year, in m (ft)

$H_{s,1\text{-yr}}$ = significant wave height with a return period of one year 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 with a return period of 100 years, in seconds

For shallow water sites, the values of $H_{100\text{-yr}}$ and $H_{1\text{-yr}}$ and associated wave periods are to be determined by measurements at site. In the event that such measurements are not available, $H_{100\text{-yr}}$ and $H_{1\text{-yr}}$ are to be approximated conservatively by the corresponding values determined above based on the Rayleigh distribution assumption or the breaking wave height at the site, whichever is less. The upper limit of wave height $H_{100\text{-yr}}$ and $H_{1\text{-yr}}$ respectively and the lower limit of wave period are related to the water depth and can be estimated following 3-3/5.3.

7.5 Reduced Wave Height (RWH)

The reduced deterministic design wave is to be applied in combination with the steady extreme wind speed model (EWM), as defined in 3-2/11.1, for 100-year or 1-year return conditions.

The wave height of the RWH model is to be expressed as a reduction factor times the extreme wave height (EWH) defined in 3-3/7.3. The reduction factor is denoted by parameter, 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 reduced deterministic wave height may be assumed as 70% of extreme wave height (EWH) or the breaking wave height at the site, whichever is less.

The amount of reduction is also to be verified such that the resultant total shear forces and/or overturning moments due to the combined RWH and EWM are not lower than those obtained by applying a combination of the extreme wave height (EWH) defined in 3-3/7.3 and the reduced extreme wind speeds (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 the wave height is approximately 0.78 times the local water depth. In deep water, a theoretical limit of wave steepness prior to breaking is 1/7.

Guidance on breaking wave hydrodynamics can be found in IEC 61400-3, Annex C.

CHAPTER 3 Environmental Conditions

SECTION 4 Currents

1 Currents

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

In the absence of site data, the speed of wind-generated surface current is to be estimated as 2%–3% of the one-hour mean wind speed at 10 m (32.8 ft) above the SWL during tropical storms and hurricanes and 1% of the one-hour mean wind speed at 10 m (32.8 ft) above the SWL during winter storms or extratropical cyclones. The direction of wind generated surface current velocity is assumed to be aligned with the wind direction.

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

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

where

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

ζ = sea floor slope, in rad

g = acceleration of gravity, in m/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 in U.S. offshore regions, 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 return period chosen for the extreme current conditions is generally not to be less than 100 years. A reduction to this requirement of return period is to be in accordance with 4-2/3.

The normal and extreme current conditions for the design of Support Structure of an offshore wind turbine are specified in 3-4/3 and 3-4/5. The Design Load Case (DLC) descriptions in 4-2/Table 1 indicate which current condition is to be applied in each DLC.

3 Normal Current Model (NCM)

The normal current model is defined as the combination of site-specific wind generated currents and applicable breaking wave induced surface currents associated with normal wave conditions. Tide and storm-generated sub-surface currents are not included.

The normal current model is to be applied in combination with normal and severe wave conditions (NSS, NWH, SSS, SWH) defined in 3-3/3 and 3-3/5.

5 Extreme Current Model (ECM)

The extreme current model is defined as the site-specific currents with return period of 100 years or one year associated with the extreme wave condition with the same return period.

The extreme current model is to be applied in combination with those extreme or reduced extreme wave conditions (ESS, EWH, RWH) defined in 3-3/7. The same return period is to be assumed for both current and waves when defining the design load cases.

CHAPTER 3 Environmental Conditions

SECTION 5 Tides, Storm Surges, and Water Levels

1 General

Tides can be classified as lunar or astronomical tides, wind tides, and pressure differential tides. The combination of the latter two is commonly called the storm surge. The water depth at any location consists of the mean depth, defined as the vertical distance between the sea floor and an appropriate near-surface datum, and a fluctuating component due to astronomical tides and storm surges. Astronomical tide variations are bounded by the highest astronomical tide (HAT) and the lowest astronomical tide (LAT). Storm surge is to be estimated from available statistics or by mathematical storm surge modeling. The still water level (SWL) referred in the definition of environmental conditions for load calculation is to be taken as the highest still water level (HSWL), which is defined as the sum of the highest astronomical level and the positive storm surge. Definitions of various water levels referred in this Guide are illustrated in 3-5/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 return period chosen for the extreme water level range is generally not to be less than 100 years. A reduction to this requirement of return period is to be in accordance with 4-2/3.

The normal and extreme water level ranges for the design of Support Structure of an offshore wind turbine are specified in 3-5/3 and 3-5/5. The Design Load Case (DLC) descriptions in 4-2/Table 1 indicate which water level range is to be applied in each DLC.

3 Normal Water Level Range (NWLR)

The normal water level range 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 strength load cases are to be performed based on the water level within the NWLR that results in the highest loads. The influence of water level variation on fatigue loads is also to be considered.

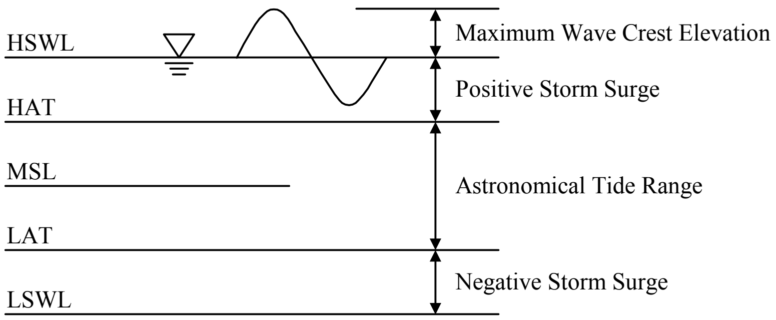
5 Extreme Water Level Range (EWLR)

The extreme water level range is to be assumed for load cases associated with extreme wave conditions with a return period of 100 years. Load calculations for strength load cases are to be performed based on the water level within the EWLR that results in the highest loads.

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 return period of 100 years
- The lowest still water level (LSWL), defined as a combination of lowest astronomical tide (LAT) and negative storm surge, with a return period of 100 years
- The water level associated with the highest breaking wave load

FIGURE 1
Definitions of Water Levels



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

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 installation.

3 Air Density

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. A commonly used value of air density as part of the normal environmental conditions for wind turbine design is 1.225 kg/m^3 (0.0765 lb/ft^3). This value is associated with an ambient air temperature range of −10°C (14°F) to –40°C (–40°F) and the relative humidity of up to 100%.

5 Ice and Snow Accumulation

For offshore wind turbines intended to be installed in areas 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 an 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 offshore wind turbine installations located in areas known to be seismically active are to be taken into account.

9.1 Levels of Earthquake Conditions

The magnitudes of the parameters characterizing the earthquakes with return periods appropriate to the design life of the Support Structure 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 Support Structure of an offshore wind turbine.

ii) *Ductility Level.* Ground motion for a rare, intense earthquake to be applied to evaluate the risk of structural collapse.

9.3 Regional and Site-specific Data

The anticipated seismicity of an area is, to the extent practicable, to be established based on regional and site specific data including, as appropriate, 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 life of the Support Structure. 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 offshore wind turbine intended to be installed in areas where ice hazards may occur, the effects of sea ice or lake ice on the Support Structure are to be taken into account in the design. Depending on the ice conditions at the site, the Support Structure may encounter with moving ice and fast ice cover..

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

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

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

13 Lightning

The lightning protection of a wind turbine is to be designed in accordance with IEC 61400-24. It is not necessary for protective measures to extend to all parts of the wind turbine, provided that safety is not compromised.

CHAPTER 4 Loads

CONTENTS

CHAPTER 4 Loads

SECTION 1 Overview

1 General

This Section pertains to the identification, definition and determination of the loads to which an offshore wind turbine installation may be subjected during and after its transportation to site and its installation. As appropriate to the planned structure, the types of loads described in 4-1/3 below are to be accounted for in the design.

3 Types of Loads to be Considered

Loads applied to an 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

Dead loads are loads which do not change during the mode of operation under consideration. Dead loads include 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, etc.).

ii) Weight of nacelle housing structure, tower, substructure, foundation, access platform, fenders, ladders, 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

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 include the following.

i) The weight of wind turbine components which can be removed.

ii) The weight of personnel and consumable supplies.

iii) The forces exerted on the wind turbine due to lifting equipment during installation and maintenance of the wind turbine.

iv) The forces exerted on the wind turbine support structure by vessels moored to the structure or accidental impact consideration for a typical supply vessel that would normally service the installation.

v) Loads associated with helicopter operations, where relevant.

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

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

Live loads occurring during transportation and installation are to be determined for each specific operation involved and the dynamic effects of such loads are to be accounted for as necessary (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

Environmental loads are loads due to the action of wind, waves, 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 of wind turbines and in accordance with the requirements specified in Chapter 3 of this Guide. The combination and severity of environmental conditions for the design of an offshore wind turbine installation 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 to the Support Structure of an offshore wind turbine from directions producing the most unfavorable effects on the 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 Environmental Loadings

1 General

Design environmental loadings are to be represented by a set of Design Load Cases (DLCs), which are defined by the 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.

As a minimum, the DLCs defined in 4-2/Table 1 are to be assessed in the design of offshore wind turbine installations. Other design load cases are to be considered, wherever they are deemed relevant to the structural integrity of a specific wind turbine design. 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.

3 Definition of Design Load Cases (DLCs)

There are three categories of design environmental conditions are specified in 4-2/Table 1, including:

i) Normal design conditions of wind turbine and appropriate normal or extreme environmental and electrical network conditions

ii) Fault design conditions of wind turbine and appropriate environmental and electrical network conditions

iii) Design conditions relevant to transportation, assembly, maintenance and repair of wind turbine and appropriate environmental and electrical network conditions

For each DLC, the "Type of Analysis" is denoted (S) for Strength or (F) for Fatigue analysis. The results of the Strength analysis are used in the structural assessment with acceptance criteria pertaining to yielding and buckling. The results of Fatigue analysis are used in the structural assessment with criteria pertaining to fatigue performance.

The DLCs indicated with S, are further classified as normal (N), abnormal (A), or transport, assembly on site, maintenance and repair (T) of an offshore wind turbine. Normal design conditions are expected to occur frequently during the lifetime of an offshore wind turbine. The corresponding operational mode of the turbine is in a normal state or with minor faults or abnormalities. Abnormal design conditions 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. The type of design conditions, N, A, or T, determines the safety factor, as specified in Chapter 5, to be applied in the structural design.

The DLCs specified in 4-2/Table 1 are generally in agreement with those required by "Table 1 – Design load cases" in IEC 61400-3 (2009). The descriptions and analysis requirements for DLCs defined in 4-2/Table 1 can be referred to Section 7.4 and Section 7.5.4 of IEC 61400-3 (2009), in conjunction with the amendments given as follows:

i) The safety factors referred in 4-2/Table 1 are specified in Chapter 5 of this Guide.

ii) The design environmental conditions referred 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.

iii) Site-specific extreme wind speeds with various combinations of return periods and averaging time durations are used to define the environmental conditions in DLC 6.1 to 6.4, DLC 7.1 and 7.2, and DLC 8.2 and 8.3 in 4-2/Table 1. This differs from IEC 61400-3 (2009) where reference is made to the wind turbine's Reference Wind Speed (V_{ref}) and the conversion factors are prescribed for different averaging time durations or return periods.

iv) The return period chosen for the extreme environmental conditions of DLC 6.1 and DLC 6.2 and for the severe wave conditions of DLC 1.6 is generally not to be less than 100 years, unless appropriate justifications are provide to ABS for a reduction and such reduction is acceptable to the governmental authorities having jurisdiction over permitting wind turbine installations. Any reduction to the return period of environmental conditions is to be subject to special considerations by ABS.

v) 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 conditions. For a site where the effect of tropical hurricanes, cyclones or typhoons needs to be considered, omni-directional wind and a yaw misalignment with ±180° is to be assumed for DLC 6.2a and 6.2b. Load calculations are to be based on the misalignment angle that results in the highest load acting on the Support Structure. The range of yaw misalignment assumed in the design of the Support Structure may be reduced to account for the contribution from an active or passive yaw control system, provided that 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 service life of an offshore wind turbine.

vi) For a site where the turbulent wind models defined in Chapter 3, Section 2 are considered insufficient to describe site-specific wind conditions, the design load cases in 4-2/Table 1 involving the combined steady wind model and deterministic design wave height are to be evaluated for the strength analysis.

vii) For those load cases, including DLC 6.1a, 6.2a, 6.3a, 7.1a, and 8.2a, which require full time domain dynamic simulations for the combined extreme turbulent wind and extreme stochastic 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 10-minute mean wind speed and significant wave height 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, provided 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 to be increased by 0.2 m/s (0.66 ft/s) relative to the value associated with 10-minute mean wind speed. Other simulation time durations can also be used along with an appropriate adjustment to the wind model and/or wave model such that the extreme responses can be adequately estimated.

viii) 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 (66 ft/s), In addition, the turbine Rated Wind Speed (V_r, see 1-4/3.41), where applicable, is to be included as one of the discrete wind speeds to be used in the load calculation.

ix) DLC 1.1 required by IEC 61400-3 (2009) for calculation of the ultimate loads acting on the Rotor-Nacelle Assembly (RNA) is not included 4-2/Table 1, which is specified for the design of the Support Structures of offshore wind turbines.

For the Support Structures of an offshore wind turbine to be installed at a site where ice is expected to occur, the DLCs specified in 4-2/Table 2 for ice conditions are to be considered.

The DLCs specified in 4-2/Table 2 for ice conditions are generally in agreement with those required by "Table 2 – Design load cases for sea ice" in IEC 61400-3 (2009). The descriptions and analysis requirements for DLCs defined in 4-2/Table 2 can be referred to Annex E of IEC 61400-3 (2009), in conjunction with the amendments described as follows:

i) The safety factors referred in 4-2/Table 2 are defined in Chapter 5 of this Guide.

ii) The design environmental conditions referred in 4-2/Table 2 for ice, wind and water level ranges are in accordance with the definitions in Chapter 3 of this Guide

iii) Site-specific extreme wind speeds with a given return period and averaging time duration are used to define the 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 100 years, unless appropriate justifications are provide to ABS for a reduction and such reduction is acceptable to the governmental authorities having jurisdiction over permitting wind turbine installations. Any reduction to the return period of environmental conditions is to be subject to special considerations by ABS.

v) Reference is also made to API RP 2N and ISO 19906 for the alternative methods of determining static and dynamic ice loads. The local ice pressure on a structural member is to be calculated using the equation below instead of the one given in Annex E.4.3.3 of IEC 61400-3 (2009):

$$\sigma_{c,local} = \sigma_c\,(5\,h_{ice}^2/A_{local} + 1)^{0.5} < 20 \text{ MPa } (2.9 \text{ ksi})$$

where

$\sigma_{c,local}$ = local ice pressure, in MPa (ksi)

σ_c = compressive strength (crushing strength) of ice, in MPa (ksi)

h_{ice} = ice thickness, in m (in)

A_{local} = area of a structural member where the local ice pressure applies, in m^2 (in^2)

TABLE 1
Design Load Cases

Design Condition	*DLC*	*Wind Condition*	*Waves*	*Wind and Wave Directionality*	*Sea Currents*	*Water Level*	*Other Conditions*	*Type of Analysis*	*Safety Factor*
1) Power production	1.2	NTM, NWP $V_{in} \le V_{hub} \le V_{out}$	NSS Joint prob. distribution of H_s, T_p, V_{hub}	COD, MUL	No current	NWLR or $\ge$ MSL		F	FDF
	1.3	ETM $V_{in} \le V_{hub} \le V_{out}$	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL		S	N
	1.4	ECD $V_{hub} = V_r - 2$ m/s (66 ft/s), V_r, $V_r + 2$ m/s (66 ft/s)	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	MIS, wind direction change	NCM	MSL		S	N
	1.5	EWS $V_{in} \le V_{hub} \le V_{out}$	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL		S	N
	1.6a	NTM, NWP $V_{in} \le V_{hub} \le V_{out}$	SSS $H_s = H_{s,SSS}$	COD, UNI	NCM	NWLR		S	N
	1.6b	NTM, NWP $V_{in} \le V_{hub} \le V_{out}$	SWH $H = H_{SWH}$	COD, UNI	NCM	NWLR		S	N
2) Power production plus occurrence of fault	2.1	NTM $V_{in} \le V_{hub} \le V_{out}$	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	Control system fault or loss of electrical network	S	N
	2.2	NTM $V_{in} \le V_{hub} \le V_{out}$	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	Protection system or preceding internal electrical fault	S	A
	2.3	EOG $V_{hub} = V_r \pm 2$ m/s (66 ft/s) and V_{out}	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	External or internal electrical fault including loss of electrical network	S	A
	2.4	NTM $V_{in} \le V_{hub} \le V_{out}$	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	No current	NWLR or $\ge$ MSL	Control, protection, or electrical system faults including loss of electrical network	F	FDF

TABLE 1
Design Load Cases

Design Condition	*DLC*	*Wind Condition*	*Waves*	*Wind and Wave Directionality*	*Sea Currents*	*Water Level*	*Other Conditions*	*Type of Analysis*	*Safety Factor*
3) Start-up	3.1	NWP $V_{in} \leq V_{hub} \leq V_{out}$	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	No current	NWLR or $\geq$ MSL		F	FDF
	3.2	EOG $V_{hub} = V_{in}$, $V_r \pm 2$ m/s (66 ft/s) and V_{out}	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL		S	N
	3.3	EDC $V_{hub} = V_{in}$, $V_r \pm 2$ m/s (66 ft/s) and V_{out}	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	MIS, wind direction change	NCM	MSL		S	N
4) Normal shut down	4.1	NWP $V_{in} \leq V_{hub}$ v V_{out}	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	No current	NWLR or $\geq$ MSL		F	FDF
	4.2	EOG $V_{hub} = V_r \pm 2$ m/s (66 ft/s) and V_{out}	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL		S	N
5) Emergency shut down	5.1	NTM $V_{hub} = V_r \pm 2$ m/s (66 ft/s) and V_{out}	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL		S	N
6) Parked (standing still or idling)	6.1a	EWM Turbulent wind model $V_{hub} = k_1 V_{10min,100\text{-}yr}$	ESS $H_s = k_2 H_{s,100\text{-}yr}$	MIS, MUL	ECM 100-yr Currents	EWLR 100-yr Water Level		S	N
	6.1b	EWM Steady wind model $V(z_{hub}) = V_{3sec,100\text{-}yr}$	RWH $H = r_2 H_{100\text{-}yr}$	MIS, MUL	ECM 100-yr Currents	EWLR 100-yr Water Level		S	N
	6.1c	RWM Steady wind model $V(z_{hub}) = V_{1min,100\text{-}yr}$	EWH $H = H_{100\text{-}yr}$	MIS, MUL	ECM 100-yr Currents	EWLR 100-yr Water Level		S	N

TABLE 1 (continued)
Design Load Cases

Design Condition	*DLC*	*Wind Condition*	*Waves*	*Wind and Wave Directionality*	*Sea Currents*	*Water Level*	*Other Conditions*	*Type of Analysis*	*Safety Factor*
6) Parked (standing still or idling) (continued)	6.2a	EWM Turbulent wind model $V_{hub} = k_1 V_{10min,100\text{-}yr}$	ESS $H_s = k_2 H_{s,100\text{-}yr}$	MIS, MUL	ECM 100-yr Currents	EWLR 100-yr Water Level	Loss of electrical network	S	A
	6.2b	EWM Steady wind model $V(z_{hub}) = V_{3sec,100\text{-}yr}$	RWH $H = r_2 H_{100\text{-}yr}$	MIS, MUL	ECM 100-yr Currents	EWLR 100-yr Water Level	Loss of electrical network	S	A
	6.3a	EWM Turbulent wind model $V_{hub} = k_1 V_{10min,1\text{-}yr}$	ESS $H_s = k_2 H_{s,1\text{-}yr}$	MIS, MUL	NCM	NWLR	Extreme yaw misalignment	S	N
	6.3b	EWM Steady wind model $V(z_{hub}) = V_{3sec,1\text{-}yr}$	RWH $H = r_2 H_{1\text{-}yr}$	MIS, MUL	NCM	NWLR	Extreme yaw misalignment	S	N
	6.4	NTM $V_{hub} \le V_{10min,1\text{-}yr}$	NSS Joint prob. distribution of H_s, T_p, V_{hub}	COD, MUL	No current	NWLR or ≥ MSL		F	FDF
7) Parked and fault conditions	7.1a	EWM Turbulent wind model $V_{hub} = k_1 V_{10min,\ 1\text{-}yr}$	ESS $H_s = k_2 H_{s,1\text{-}yr}$	MIS, MUL	NCM	NWLR		S	A
	7.1b	EWM Steady wind model $V(z_{hub}) = V_{3sec,\ 1\text{-}yr}$	RWH $H = r_2 H_{1\text{-}yr}$	MIS, MUL	NCM	NWLR		S	A
	7.1c	RWM Steady wind model $V(z_{hub}) = V_{1min,\ 1\text{-}yr}$	EWH $H = H_{1\text{-}yr}$	MIS, MUL	NCM	NWLR		S	A
	7.2	NTM $V_{hub} \le V_{10min,1\text{-}yr}$	NSS Joint prob. distribution of H_s, T_p, V_{hub}	COD, MUL	No current	NWLR or ≥ MSL		F	FDF
8) Transport, assembly, maintenance and repair	8.1	To be defined by the manufacturer and Operator						S	T
	8.2a	EWM Turbulent wind model $V_{hub} = k_1 V_{10min,1\text{-}yr}$	ESS $H_s = k_2 H_{s,1\text{-}yr}$	COD, UNI	NCM	NWLR		S	A
	8.2b	EWM Steady wind model $V(z_{hub}) = V_{3sec,1\text{-}yr}$	RWH $H = r_2 H_{1\text{-}yr}$	COD, UNI	NCM	NWLR		S	A
	8.2c	RWM Steady wind model $V(z_{hub}) = V_{1min,1\text{-}yr}$	EWH $H = H_{1\text{-}yr}$	COD, UNI	NCM	NWLR		S	A
	8.3	NTM $V_{hub} \le V_{10min,1\text{-}yr}$	NSS Joint prob. distribution of H_s, T_p, V_{hub}	COD, MUL	No current	NWLR or ≥ MSL	No grid during installation period	F	FDF

TABLE 1 (continued)
Design Load Cases

Notes:

The symbols and abbreviations used in the table are summarized as follows.

COD	co-directional (aligned) wind and wave direction
DLC	design load case
ECD	extreme coherent gust with direction change (3-2/11.11)
ECM	extreme current model (3-4/5)
EDC	extreme direction change (3-2/11.9)
EOG	extreme operating gust (3-2/11.5)
ESS	extreme sea state (3-3/7.1)
EWH	extreme wave height (3-3/7.3)
EWLR	extreme water level range (3-5/5)
EWM	extreme wind speed model (3-2/11.1)
EWS	extreme wind shear (3-2/11.13)
MIS	misaligned wind and wave directions
MSL	mean sea level (3-5/Figure 1)
MUL	multi-directional wind and wave
NCM	normal current model (3-4/3)
NTM	normal turbulence model (3-2/9.3)
NWH	normal wave height (3-3/3.3)
NWLR	normal water level range (3-5/3)
NWP	normal wind profile model (3-2/9.1)
NSS	normal sea state (3-3/3.1)
RWH	reduced wave height (3-3/7.5)
RWM	reduced wind speed model (3-2/11.3)
SSS	severe sea state (3-3/5.1)
SWH	severe wave height (3-3/5.3)
UNI	uni-directional wind and wave directions

F	fatigue (4-2/3)
S	strength (4-2/3)
N	normal (4-2/3)
A	abnormal (4-2/3)
T	transport, assembly, maintenance and repair (4-2/3)
FDF	fatigue design factor (Chapter 5)
H	deterministic design wave height
H_s	significant wave height
$H_{s,1\text{-yr}}$, $H_{s,100\text{-yr}}$	significant wave heights with return period of 1-year and 100-year, respectively (Chapter 3, Section 3)
$H_{1\text{-yr}}$, $H_{100\text{-yr}}$	maximum wave heights with return period of 1-year and 100-year, respectively (Chapter 3, Section 3)
k_1	simulation time scaling factors for 10-minute mean wind speed (4-2/3)
k_2	simulation time scaling factors for significant wave height (4-2/3)
r_2	reduction factor for extreme wave height (3-3/7.5)
T_p	peak period of wave spectrum
V_{hub}	10-minute mean wind speed at hub height
$V(z_{hub})$	steady wind speed at hub height, z_{hub}
V_{in}	cut-in wind speed (1-4/3.5)
V_{out}	cut-out wind speed (1-4/3.7)
$V_r \pm 2$ m/s (66 ft/s)	sensitivity to the wind speeds in the range is to be analyzed (4-2/3)
$V_{10\text{min},100\text{-yr}}$, $V_{1\text{min},100\text{-yr}}$, $V_{3\text{sec},100\text{-yr}}$	100-year return wind speed at hub height with various averaging time durations (Chapter 3, Section 2)
$V_{10\text{min},1\text{-yr}}$, $V_{1\text{min},1\text{-yr}}$, $V_{3\text{sec},1\text{-yr}}$	1-year return wind speed at hub height with various averaging time durations (Chapter 3, Section 2)

TABLE 2
Design Load Cases for Ice Conditions

Design Condition	*DLC*	*Ice Condition*	*Wind Condition*	*Water Level*	*Type of Analysis*	*Safety Factor*
Power Production	E1	Horizontal load from temperature fluctuations	NTM $V_{hub} = V_r \pm 2$ m/s (66 ft/s) and V_{out} Wind Speed resulting in maximum thrust	NWLR	S	N
	E2	Horizontal load from water fluctuations or arch effect	NTM $V_{hub} = V_r \pm 2$ m/s (66 ft/s) and V_{out} Wind Speed resulting in maximum thrust	NWLR	S	N
	E3 For extrapolation of extreme events	Horizontal load from moving ice floe at relevant velocities $h_{ice} = h_{ice,\ 100\text{-}yr}$ in open sea $h_{ice} = h_{ice,\ m}$ for land-locked waters	NTM $V_{hub} = V_r \pm 2$ m/s (66 ft/s) and V_{out} Wind Speed resulting in maximum thrust	NWLR	S	N
	E4	Horizontal load from moving ice floe at relevant velocities $h_{ice} = h_{ice,\ 100\text{-}yr}$ in open sea $h_{ice} = h_{ice,\ m}$ for land-locked waters	NTM $V_{in} \le V_{hub} \le V_{out}$	NWLR	F	FDF
	E5	Vertical force from fast ice covers due to water level fluctuations	No wind load applied	NWLR	S	N
Parked	E6	Pressure from hummocked ice and ice ridges	EWM $V_{hub} = V_{10\text{min},\ 1\text{-yr}}$	NWLR	S	N
	E7	Horizontal load from moving ice floe at relevant velocities $h_{ice} = h_{ice,\ 100\text{-}yr}$ in open sea $h_{ice} = h_{ice,\ m}$ for land-locked waters	NTM $V_{hub} \le V_{10\text{min},1\text{-yr}}$	NWLR	F	FDF

Notes:

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

h_{ice} ice thickness

$h_{ice,100\text{-}yr}$ 100-yr return ice thickness

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

CHAPTER 4 Loads

SECTION 3 Determination of Environmental Loads

1 General

Model or on-site test data may be employed to establish environmental loads. Alternatively, environmental loads may 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 proven sufficiently accurate in practice, and it is shown to be appropriate to the structure's characteristics and site conditions. The calculation methods presented herein are offered as guidance representative of current acceptable methods.

3 Wind Loads

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

Aerodynamic loads induced by airflow 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.

For offshore wind turbines installed in a wind farm, the 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 effects from neighboring offshore wind turbines.

For wind drag loads 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 turbine support structure, the following relation is to be used:

$$F_w = (\rho/2g)C_s A V^2$$

where

F_w = wind drag load, in N (lb)

ρ = 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 = projected area of member on a plane normal to the direction of the considered force, in m^2 (ft^2)

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

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.

In the absence of experimental data, values for the shape coefficient (C_s) are to be assumed as follows.

TABLE 1
Values of C_s

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

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

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

Where applicable, cyclic loads due to vortex induced vibration of the Support Structure 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 where applicable. This would especially apply to fabrication or transportation phases.

5 Wave Loads

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

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

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

$$F = F_D + F_I$$

where

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

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

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

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

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

where

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

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

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

where

C_M	=	inertia coefficient based on the displaced mass of fluid per unit length
a_n	=	component of the fluid acceleration vector normal to the axis of the member, in m/s^2 (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.

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

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

where

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

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

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

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

For installation sites where the ratio of water depth to wave length is less than 0.25, nonlinear effects of wave action are to be taken into account. This may be done by modifying linear diffraction theory to account for nonlinear effects or by performance of model tests.

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. Where appropriate, flutter and dynamic amplification due to vortex shedding are to be taken into account.

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

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

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

where

F_L = total lift force per unit length, in N/m (lb/ft)

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

ρ = weight density of water, in N/m^3 (lb/ft^3)

g = acceleration of gravitation, in m/s^2 (ft/s^2)

A = projected area per unit length in a plane normal to the direction of the force, in m^2/m (ft^2/ft)

C_L = lift coefficient

C_D = drag coefficient (see 4-3/5)

U = local current velocity, in m/s (ft/s)

In general, lift force may become significant for long cylindrical members with large length-diameter ratios and is to be checked in the design under these conditions. The source of CL values employed is to be documented.

9 Ice and Snow Accumulation Induced Loads

At locations where offshore wind turbines are subject 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 offshore wind turbine installations 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 structure. 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 given in the API RP 2A-WSD, is used for structural analysis, input values of ground motion (spectral acceleration representation) to be used are not to be less severe than the following.

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

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

If time history method is used for structural analysis, at least three sets of ground motion time histories are to be employed. The 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 design is to reflect the extent of and interval between cleaning of submerged structural parts.

15 Ice Loads

Ice loads acting on an offshore wind turbine are both static and dynamic loads. Static loads are normally generated by temperature fluctuations or changes in water level in a fast ice cover. Dynamic loads are caused by moving ice interactions with the Support Structure.

The methods to determine ice loading on the Support Structure of an offshore wind turbine are described in Annex E of IEC 61400-3 (2009). Further reference is made to API RP 2N and ISO 19906.

The global forces exerted by ice on the turbine 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 turbine support structure need to be considered. Possible ice jamming between legs is to be accounted for where the Support Structure is designed to consist of multiple legs.

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

CHAPTER 5 Structure and Foundation Design

CONTENTS

CHAPTER 5 Structure and Foundation Design

SECTION 1 General Design Requirements

1 Overview

This Section outlines general concepts and considerations to be incorporated in the design of the Support Structure of an offshore wind turbine. The subsequent Sections 2 through 4 of Chapter 5 dealing specifically with the steel, concrete and foundation design are to be viewed in light of the requirements specified in this Section.

Wherever references in Chapter 5 are made to the API RP 2A, if applicable, other 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 service life extension and reuse of the existing Support Structure of an offshore wind turbine is to be in accordance with Chapter 4, Section 1 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 Analytical Approaches

3.1 Format of Design Approach

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.

Use of other formats of design approach is subject to special considerations by ABS. It is designer's responsibility to demonstrate that the adopted alternative approach can result in a design with a safety level equivalent to, or exceeding that of, the design using the approach described in this Guide.

3.3 Loading Formats

With reference to Chapter 3 and 4, both stochastic and deterministic formats are to be used to establish design load effects. Where relevant, consideration is to he given to the effect of dynamic amplification. The influence of the less severe environmental loads in combination with the operational loads is to be investigated for their potential to produce maximum peak stresses due to resonance with the Support Structure of an offshore wind turbine.

When considering an earthquake in seismically active areas (see 3-6/9 and 4-3/11), a dynamic analysis is to be performed using the recognized methods, such as those given in API RP 2A.

A dynamic analysis is also to be considered to assess the effects of environmental or other types of loads where dynamic amplification is expected. When a fatigue analysis is performed, 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.

3.5 Combination of Loading Components

Loads imposed during and after installation are to be taken into account. Loading combinations are to reflect the design environmental loadings as specified in 4-2. If site-specific directional data are not available, the direction of applied environmental loads is to be determined to produce the most unfavorable influence on the Support Structure. Reference is made to Chapter 5, Sections 2, 3 and 4 for the minimum load combinations to be considered.

5 Overall Design Considerations

5.1 Design Life

The design life of the Support Structure of an offshore wind turbine is to be not less than 20 years.

Continuance of classification beyond the design life is to be subject to the survey requirements specified in 1-3/1.19 and engineering analyses.

5.3 Air Gap

A minimum air gap of 1.5 m (5 ft) is to be provided between the 100-year return maximum wave crest elevation and the lowest edge of the Support Structure for which wave forces are not included in the design. After accounting for the initial and expected long-term settlements of the structure, the design wave crest elevation is to be superimposed on the Still Water Level (SWL, see 3-5/Figure 1). Additional consideration is to be given to wave run-up, tilting of the structure and, where appropriate, tsunamis.

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.

For Offshore Wind Turbine Installations in U.S. offshore regions, an additional air gap of 15% of the 100-year maximum wave crest elevation, as required in API Bulletin 2INT-MET, is to be applied to account for the local random wave crest. When this additional air gap is not included, the portion of the Support Structure and any appurtenances and localized components that lie below the 100-year return maximum crest elevation are to be designed to withstand the local wave force associated with the local random wave crest. Such local wave force is not to be taken into account when establishing the global loads applied to the Support Structure.

5.5 Structural Dynamic Properties

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 the rotor speed, blade passing frequency and design wave periods. 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 offshore wind turbine’s service life may change the natural frequencies of the Support Structure and are to be suitably taken into account in the design.

5.7 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

5.9 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. 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.

5.9.1 Submerged Zone

That part of the Support Structure of an offshore wind turbine below the splash zone.

5.9.2 Splash Zone

That part of the Support Structure containing the areas above and below the Still Water Level (SWL) and regularly subject to wetting due to wave and tide actions. Characteristically, the splash zone is not easily accessible for field painting, nor protected by cathodic protection.

5.9.3 Atmospheric Zone

That part of the Support Structure of an offshore wind turbine above the splash zone.

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 Guides.

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 in the structural analysis.

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) The natural period of the Support Structure is to be checked to verify that it 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

Considerations are 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 sulphate attack is anticipated, mitigation measures are to be implemented by choosing appropriate cements, incorporating pozzolans in the mix, or applying suitable coatings on the surfaces.

v) Instability of structural members due to submersion is to be considered, with due account for second-order effects produced by factors such as geometrical imperfections.

vi) Where necessary, protection against horizontal sliding along the sea floor is to be provided by means of skirts, shear keys or equivalent means.

vii) The long-term resistance to abrasion, cavitation, freeze-thaw durability and strength retention of the concrete are to be considered. Means are to be provided to minimize reinforcing steel corrosion.

viii) Provision is to be made to maintain adequate negative buoyancy at all times to resist the uplift forces from wind, waves, currents, and overturning moments.

7.7 Concrete-Steel Hybrid Foundation Structures

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

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

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

7.9 Self-Elevating Unit to be Used as the Support Structure

A self-elevating mobile offshore unit converted to the site dependent Support Structure of an offshore wind turbines 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 spud cans, the legs may be assumed pinned at the reaction point. For mat supported units, the soil structure interaction may be modeled using discrete elastic elements (springs).

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

Units with spud cans 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 Overview

The requirements of this Section are to be applied in the design and analysis of the steel Support Structure of an offshore wind turbine. 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 the 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 is to be considered subject to a review of the specifications of alternative materials and the proposed methods of fabrication.

1.3 Corrosion Protection

Materials are to be protected from the effects of corrosion by the use of a corrosion protection system including the use of 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 General Design Criteria

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. The effects of a minimum set of loading conditions, as specified in 5-2/5, on the Support Structure are to be determined, and the resulting structural responses of the Support Structure are not to exceed the safety and serviceability criteria given in 5-2/9.

The use of design methods and associated safety and serviceability criteria, other than those specified in this Section, is permitted. It is designer's responsibility to demonstrate that the use of such alternative methods can result in a structure which possesses a level of safety equivalent to or exceeding that provided by applying the requirements in this Section.

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

5 Loading Conditions

Loadings that produce the most unfavorable effects on the structure during construction, transportation, installation and during its service life are to be considered. Loadings to be investigated for conditions after installation are to include at least those relating to both the realistic operating and environmental conditions combined with dead and live loads (see Chapter 4, Section 1) that are appropriate to the function and operations of an offshore wind turbine.

Loading combinations are to reflect the design environmental loadings as specified in Chapter 4, Section 2. The Design Load Cases in 4-2/Table 1 are to be assessed as a minimum requirement of design environmental conditions. Additional Design Load Cases for ice conditions are to be considered in accordance with 4-2/3 for an offshore wind turbine to be installed at a site where ice is expected to occur.

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 and live loads appropriate to the offshore wind turbine operation and function which may be occurring at the onset of an earthquake.

7 Structural Analysis

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 effects are to be accounted for if the wind and wave energy in the frequency range of the natural frequencies of the Support Structure is of sufficient magnitude to produce significant dynamic responses in the structure. In assessing the need for dynamic analyses, information regarding the natural frequencies of the Support Structure in its intended position is to be obtained.

iii) Where dynamic analyses are required, the interactions between wind and wave loadings and between external loads and structural response are to be assessed based on structural analyses in the time domain for the simultaneous application of time series of wind loads and wave loads. Other rational methods for calculating loadings are acceptable provided that the designer can demonstrate that the adopted method leads to an equivalent design. When structural analyses are performed separately for wind and wave loads, the designer is to verify that dynamic interaction is suitably accounted for.

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 idealized based on the 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) In an ultimate strength analysis, it is to be demonstrated that the collapse mode (mechanism) which corresponds to the smallest loading intensities has been used for the determination of the ultimate strength of the structure. Buckling and other destabilizing nonlinear effects are to be taken into account in the plastic analysis. Whenever non-monotonic or repeating loads are present, it is to be demonstrated that the structure will not fail by incremental collapse or fatigue.

vii) Under dynamic loads, when plastic strains may occur, the considerations specified in *v)* are to be satisfied and any buckling and destabilizing nonlinear effects are to be taken into account.

9 Strength Design Criteria

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 for the same structural component in the Support Structure.

Reference is also made to 5-1/3.1 for the general design requirements.

9.3 Working Stress Design (WSD) Approach

When the strength design of the steel Support Structure is based on the WSD approach (see 5-1/3.1), the design acceptance criteria are to be expressed in terms of appropriate basic allowable stresses in accordance with the requirements specified in this Subsection. Linear, elastic methods can be employed in the analysis of the 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 load combination is to be in accordance with 5-2/5. 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, assembly on site, maintenance and repair of offshore wind turbines, as defined in Chapter 4, Section 2.

9.3.1 Individual Stresses in Structural Members

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

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

where

$F_{allowable}$ = allowable stress

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

$F.S.$ = factor of safety

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

  = 1.5 for axial or bending stress

  = 2.5 for shear stress

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

  = 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 of the Support Structure (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 Subject to a Single Action

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

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

where

$F_{allowable}$ = allowable stress

F_{cr} = critical buckling strength of a structural member subject to axial compression or critical bending strength of a structural member subject to 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)

  = $1.5/\psi$

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

  = $1.25/\psi$

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

  = $1.67/\psi$

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

9.3.3 Structural Members Subject to Combined Axial Load and Bending

Structural members subject to 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 safety factors, which are the reciprocal of corresponding utilization factors, specified in 5-2/9.3.2.

9.3.4 Allowable Stress of Plated Structures

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

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

where

$F_{allowable}$ = allowable stress

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

$F.S.$ = factor of safety

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

  = 1.33

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

  = 1.11

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

  = 1.5

9.3.5 Buckling Strength of Plated Structures

The buckling strength of plated structures is to be designed according to the ABS *Guide for Buckling and Ultimate Strength Assessment for Offshore Structures* in conjunction with the safety factors, which are the reciprocal of corresponding utilization 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 (γ_f) for loads, as specified in 5-2/Table 1, are to be applied to the aggregate dead loads, maximum live loads and design environmental loads.

When a dead load or live 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
Partial Safety Factors (γ_f) for Loads

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

Note: The design conditions represented by N, A and T are defined in 4-2/3 and 4-2/Table 1.

For cylindrical member and connection design, the resistance factors and strength criteria are to be determined in accordance with API RP 2A-LRFD. For the structural members with other types of sections within the scope of the AISC *Steel Construction Manual* (LRFD part), the resistance factors and member strength are to be determined based on the relevant design requirements specified in that standard. For the ultimate capacity of flat plate and the cylindrical shell structure, reference may be made to the ABS *Guide for Buckling and Ultimate Strength Assessment for Offshore Structures* and a resistance factor of $1.05/\psi$, where ψ is the adjustment factor as defined in 5-2/9.3.2, is to be applied.

11 Structural Response to Earthquake Loads

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.

In U.S. offshore regions, the general design requirements for earthquake are to be in accordance with API RP 2A-WSD. In other seismically active locations around the world, a seismic report is to be submitted.

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 special studies.

13 Fatigue Assessment

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, an assessment of fatigue life is to be carried out. Emphasis is to be given to joints and members that are difficult to inspect and repair once the Support Structure is in service and those susceptible to corrosion-accelerated fatigue.

For structural members and joints that require a detailed assessment of cumulative fatigue damage, the calculated fatigue life is to be greater than the design life of the offshore wind turbine installation times the safety factors for fatigue life [i.e., fatigue design factors (FDFs)] as defined in 5-2/Table 2 below.

The fatigue resistance of structural details is to be evaluated in accordance with ABS *Guide for Fatigue Assessment of Offshore Structures*.

The load combination for fatigue assessment is to be in accordance with 5-2/5. A minimum set of Design Load Cases (DLCs) for fatigue assessment is specified in 4-2/Table 1, where “F” in the column titled as “Type of Analysis” designates the fatigue assessment. In the case that the design is based on the LRFD approach, the load factors for all fatigue loads are to be taken as 1.0 in the fatigue assessment.

The dynamic effect is to be suitably taken into consideration in the fatigue assessment. In general, the time series of fatigue loadings (stresses) are to be calculated based on structural analyses in the time domain for the simultaneous application of time series of wind loads and wave loads. Other rational methods for calculating fatigue loadings are acceptable provided that the designer can demonstrate that the adopted method leads to an equivalent design obtained using the methods described above.

TABLE 2
Safety Factors for Fatigue Life of Structures (Fatigue Design Factors)

Importance	*Inspectable and Repairable*	
	Yes	*No*
Non-Critical	1	3
Critical	3	5

Notes:

1 “Critical” indicates that failure of these structural items would result in the rapid loss of structural integrity and produce an event of unacceptable consequence.

2 A Fatigue Design Factor of 1.0 is applicable to

- inspectable and repairable non-critical structural members above the splash zone
- diver or ROV inspectable and repairable redundant framing

3 For the turbine tower structure installed above the splash zone, a Fatigue Design Factor of 2.0 may be applied provided that the tower structure is inspected at times of anticipated scheduled survey or when structural damage is suspected such that critical crack development can be detected and repaired.

15 Stresses in Connections

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

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

17 Structure-Pile Connections

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

In the design of the grouted pile to structure connection, considerations are 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 grouting shrinkage. Adequate clearance between the pile and the sleeve or jacket leg 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 during installation.

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

For grouted pile connections undertaking axial loads, general references are to be made to API RP 2A. Special attentions are to be paid to the limitation of geometric configuration required by the design criteria in API RP 2A. For grouted pile connections whose geometries are not covered by the existing design criteria, special considerations are 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 those grouted pile connections expected to undertake bending moments, their strengths are to be assessed by suitable analysis methods or by tests. The assessment results are to be submitted for review.

For 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. 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 water depths at which they are expected to be located. Hydrostatic collapse is to be checked in accordance with the ABS *Guide for Buckling and Ultimate Strength Assessment for Offshore Structures* or API RP 2A if applicable.

21 Deflections

The Support Structure deflections that may affect 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 accounted for in analysis.

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 Overview

The requirements of this Section are to be applied to the offshore wind turbine Support Structures 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 is to be specially considered. Specifications for alternative materials, details of the proposed methods of manufacture and, where available, evidence of satisfactory previous performance, are to be submitted for approval.

For structural lightweight concrete, the reference is made to ACI 213R; lightweight aggregates is 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. Problems to be specifically addressed include chemical deterioration of concrete, corrosion of the reinforcement and hardware, abrasion of the concrete, freeze-thaw durability, and fire hazards as they pertain to the zones of exposure defined in 5-1/5.9.

Test mixes is to be prepared and tested early in the design phase to verify that proper values of strength, creep, alkali resistance, etc. will 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 General Design Criteria

3.1 Design Method

3.1.1 General

The requirements of this Section relate to the ultimate strength method employing the Limit State Design, or Load and Resistance Factor Design (LRFD), format of design.

3.1.2 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_kF_k).

3.1.3 Design Strength

In the analysis of sections, the design strength of a given material is obtained by multiplying the material strength, f_k, by the appropriate strength reduction factor, φ (i.e., design strength = φ f_k).The material strength, f_k, for concrete is the specified compression strength of concrete (f_c') after 28 days and for steel is the minimum specified yield strength (f_y).

3.3 Load Definition

The load categories referred in this Section (i.e., dead loads, live loads, deformation loads, and design environmental loads) are defined in 4-1/3. The load combinations are defined in 5-3/5.3.

Loading combinations are to reflect the design environmental loadings as specified in Chapter 4, Section 2. The Design Load Cases in 4-2/Table 1 are to be assessed as a minimum requirement of design environmental conditions. Additional Design Load Cases for ice conditions are to be considered in accordance with 4-2/3 for an offshore wind turbine to be installed at a site where ice is expected to occur.

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.

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 Required Strength (Load Combinations)

The load combination is to be in accordance with 5-2/5 in general. 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 given above, the other simultaneously occurring load combinations during construction, transport and installation phases are to be considered if they can cause critical load effects.

5.5 Strength Reduction Factors

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

i) In the case of bending without axial tension, φ = 0.90

ii) In the case of axial compression or axial compression combined with bending.

- For reinforced members with spiral reinforcement, φ = 0.70
- For other reinforced members (excluding slabs and shells), φ = 0.65

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

  f'_c = specified compression strength of concrete

  A_g = gross area of section

  P_u = axial design load in compression member

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

- For slabs and shells, φ = 0.70

iii) In the case of shear and torsion, φ = 0.75

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

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

5.7 Fatigue

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 subject to cyclic tensile stresses greater than 50% of the allowable static stress, the lap length or prestressing development length is to be increased by 50%.

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

Particular attention is to be given to submerged areas subject 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 subject to corrosive action.

Where an analysis of the fatigue life is performed for the concrete Support Structure of an offshore wind turbine, 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 offshore wind turbine. In order to estimate the cumulative fatigue damage under variable amplitude stresses, a recognized cumulative rule is to be used. Miner's rule is an acceptable method for the cumulative fatigue damage analysis..

5.9 Serviceability Requirements

5.9.1 Serviceability

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, φ, and partial safety factors for loads, c_k, are to be taken as 1.0. The unfactored (c_k =1.0) load combination of most unfavorable dead loads, deformation loads and live loads as well as the design environmental loads is to be applied.

Using this method, the reinforcing stresses are to be limited in compliance with 5-3/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.

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

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

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

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

Stage	*Loading*	*Allowable Stress, MPa (ksi)*	
		Reinforcing Steel, f_s	*Prestressing Tendons, D_{ps}*
Construction: where cracking during construction would be detrimental to the completed structure	All loads on the structure during construction	160 (23.0)	130 (18.5)
Construction: where cracking during construction is not detrimental to the completed structure	All loads on the structure during construction	210 (30.0) or $0.6 f_y$, whichever is less	130 (18.5)
Transportation and installation	All loads on the structure during transportation and installation	160 (23.0)	130 (18.5)
At offshore site	Dead and live loads plus design environmental loads	$0.8 f_y$	

f_y = yield stress of the reinforcing steel

f_s = allowable stress in the reinforcing steel

D_{ps} = increase in tensile stress in prestressing steel with reference to the stress at zero strain in the concrete.

7 Analysis and Design

7.1 General

Generally, the analysis of the concrete Support Structure of an offshore wind turbine may be performed under the assumptions of linearly elastic materials and linearly elastic structural behavior, following 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 concrete Support 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 that 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 subject 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 × 10^3 MPa (29 × 10^3 ksi). The modulus of elasticity of prestressing tendons is to be determined by tests.

7.3.8 Uniaxial Tension – Reinforcement

The stress-strain relation of non-prestressed steel reinforcement in uniaxial tension is to be assumed as shown in 5-3/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 subject to external pressure and the possibility of their collapse (implosion) by failure of concrete in compression.

7.7 Deflection Analysis

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

7.9 Analysis and Design for Shear and Torsion

The applicable requirements of ACI 318 or their equivalent are to be complied with in the analysis and design of members subject 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 subject 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

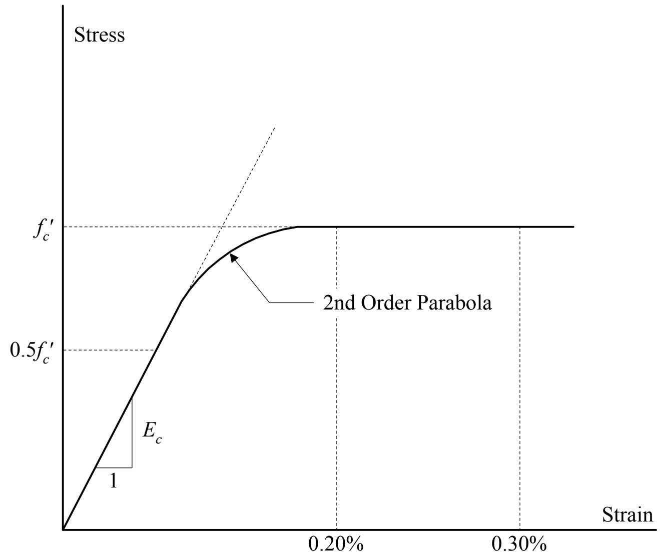
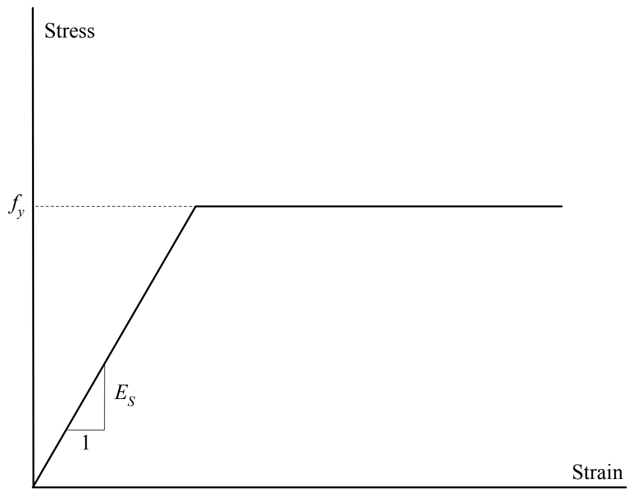


FIGURE 2
Idealized Stress-Strain Relation for Non-Prestressed Steel in Uniaxial Tension

$E_S = 200 \times 10^3$ MPa (29×10^3 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 analysis is to be performed to determine the response of the structure to design earthquake loading. The Support Structure of an offshore wind turbine 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 experience 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 taking into account:

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

The dynamic analysis for earthquake loadings may be performed by any recognized method, such as determination of time histories of the response by 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-linearities of the structural model. The possibility of dynamic instability (dynamic buckling) of individual members and of the whole structure is to be considered.

7.15 Seismic Design

7.15.1 Compressive Strain

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

7.15.2 Flexural Bending or Load Reversals

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

7.15.3 Web Reinforcement

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

i) The diameter of rods used as stirrups is not to be less than 10 mm (#3 bar)

ii) Only closed stirrups (stirrup ties) are to be used. T-headed bars or other mechanically headed bars may be used if their effectiveness has been verified.

ii) 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 above, from a plastic hinge. Lap splices are to be designed in accordance with ACI 318. Mechanical and welded splices are permitted. Mechanical splices are to be in compliance with ACI 349.

9 Design Details

9.1 Concrete Cover

9.1.1 General

The following minimum concrete cover for reinforcing bars is required:

i) Atmospheric zone not subject to salt spray: 50 mm (2 in.)

ii) Splash and atmospheric zones subject to salt spray and exposed to soil: 65 mm (2.5 in.)

iii) Submerged zone: 50 mm (2 in.)

iv) Areas not exposed to weather or soil: 40 mm (1.5 in.)

v) Cover of stirrups may be 13 mm (0.5 in) less than covers listed above

9.1.2 Tendons and Ducts

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

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

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

i) 1.5 times the nominal aggregate size

ii) 1.5 times the maximum diameter of reinforcement, or 19 mm (0.75 in.)

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

9.3 Minimum Reinforcement

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

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

where

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

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

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

9.5 Reinforcement Details

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

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

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

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

9.7 Post Tensioning Ducts

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

The inside diameter of ducts is to be at least 6 mm (0.25 in.) larger than the diameter of the post-tensioning tendon 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 the concrete at the time of placing is to be at least 4°C (40°F) and the concrete is to be maintained at this or a higher temperature until it has reached a strength of at least 5 MPa (700 psi).

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

11.3.3 Hot Weather

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

11.3.4 Curing

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

11.3.5 Sea Water

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

11.3.6 Temperature Rise

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

11.3.7 Joints

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

11.3.8 Watertight Joints

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

11.5 Reinforcement

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

11.7 Prestressing Tendons, Ducts, and Grouting

11.7.1 General

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

11.7.2 Cleanliness

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

11.7.3 Storage

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

11.7.4 Protective Coatings

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

11.7.5 Entry of Water

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

Site investigations, design considerations for the supporting soil and the influence of the soil and foundation on the Support Structure of an offshore wind turbine are covered in this Section. 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 are quite variable, 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 structure is to be placed
- 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 5-4/1, 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 to verify that the data obtained are pertinent to the actual location of the Support Structures of offshore wind turbines.

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 Support Structures.

i) Soundings or contours of the sea bed

ii) Position of bottom shapes which might affect scour

iii) The presence of boulders, obstructions, and small craters

iv) Gas seeps

v) Shallow faults

vi) Slump blocks

vii) Ice scour of sea floor sediments

viii) Subsea permafrost or ice bonded soils

5.5 Geological Survey

Data of the regional geological characteristics which can affect the design and siting of the Support Structure 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 bore hole, for either individual or clustered piles, is to be the design penetration plus a zone of influence. The zone of influence is to be at least 15.2 m (50 ft) or 1.5 times the diameter of the cluster, whichever is greater, unless it can be demonstrated by analytical methods that a lesser depth is justified. Additional bore holes of lesser depth are required if discontinuities in the soil are likely to exist within the area of the structure.

For a gravity-type foundation, the required depth of at least one boring is to be at least equal to the larger horizontal dimension or three times the smaller dimension of the base, whichever is greater. In-situ tests are to be carried out, where possible, to a depth that will include the anticipated shearing failure zone.

A reasonably continuous profile is to be obtained during recovery of the boring samples. The recovery of the materials to a depth of 12 m (40 ft) below the mudline is to be as complete as possible. Thereafter, samples at significant changes in strata are to be obtained, at approximately 3 m (10 ft) intervals to 61 m (200 ft) and approximately 8 m (25 ft) intervals below 61 m (200 ft).

The existence of carbonate soils is to be determined in frontier areas or areas known to contain carbonate materials during the subsurface investigation. Additional field and laboratory testing may be required when a soil profile shows the contained carbonate materials are more than 15% of the soil fraction.

5.9 Soil Testing Program

The testing program is to reveal the necessary engineering properties of the soil including strength, classification and deformation properties of the soil. Testing is to be performed in accordance with recognized standards.

At least one undrained strength test (vane, drop cone, unconfined compression, etc.) on selected recovered cohesive samples is to be performed in the field.

Where practicable, a standard penetration test or equivalent on each significant sand stratum is to be performed, recovering samples where possible. Field samples for laboratory work are to be retained and carefully packaged to minimize changes in moisture content and disturbance.

Samples from the field are to be sent to a recognized laboratory for further testing. They are to be accurately labeled and the results of visual inspection recorded. The testing in the laboratory is to include at least the following.

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

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

iii) Determine density of selected samples.

iv) As necessary, develop appropriate constitutive parameters or stress-strain relationships from either unconfined compression tests, unconsolidated undrained triaxial compression tests, or consolidated undrained triaxial compression tests.

v) Perform grain size sieve analysis, complete with percentage passing 200 sieve, on each significant sand or silt stratum.

vi) Other advanced testing to determine soil parameters necessary for detailed geotechnical analyses.

For pile-supported structures, consideration is also to be given to the need for additional tests to adequately describe the dynamic, creep and set-up characteristics of the soil as well as the static and cyclic lateral properties of soil-pile system.

For gravity structures, laboratory tests are also to include, where necessary, the following.

i) Shear strength tests with pore pressure measurements. The shear strength parameters and pore-water pressures are to be measured for the relevant stress conditions

ii) Cyclic loading tests with deformation and pore pressure measurements to determine the soil behavior during alternating stress

iii) Permeability and consolidation tests performed as required

7 Foundation Design Requirements

7.1 General

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 Support Structure of an offshore wind turbine.

The soil and the Support Structure are to be considered as an interactive system; the results of analyses, as required in subsequent paragraphs, are to be evaluated from this point of view.

In addition to this Subsection, specific design requirements are described in 5-4/9 for pile foundations and 5-4/11 for gravity structures. Other types of foundations are subject to special considerations by ABS.

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 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 of the Support Structure of an offshore wind turbine may be accomplished by lumped parameter, foundation impedance functions, or by continuum approaches including the use of finite element methods. Such models are to include consideration of the internal and radiational damping provided by the soil and the effects of soil layering.

Studies of the dynamic response of the structure are to include, where applicable, consideration of the nonlinear and inelastic characteristics of the soil, the possibilities of deteriorating strength and increased or decreased damping due to cyclic soil loading, and the added mass of soil subject to acceleration. Where applicable, the influence of nearby structures is to be included in the analysis.

7.13 Loading Conditions

Those loading conditions which produce the worst 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 design environmental conditions, as specified in Chapter 4, Section 2, combined with dead loads and live 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 to verify that structure's function and safety 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 structure.

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

i) Cyclic, creep and soil set-up effects on the soil strength.

ii) Punch through failure in layered soils.

iii) Installation tolerances and possible soil disturbance due to protuberances as applicable.

iv) If the reference site is within a seismic zone, potential of soil liquefaction and its impact.

v) Pile group effect if applicable.

vi) Bottom instability such as slope stability, hydraulic stability (seepage, piping), scour potential, etc.

Where applicable, the effects of close spacing on the load and deflection characteristics of pile groups are to be determined. The allowable load for a group, both axial and lateral, is not to exceed the sum of the apparent individual pile allowable loads reduced by a suitable factor.

The required load conditions and strength criteria of steel structural members are to be in accordance with 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 offshore wind turbine Support Structure.

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 is to consist 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 found in the API RP 2A, or another method 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 pile are evaluated using the above cited API method, the allowable values of axial pile bearing and pullout loads are to be determined by dividing the ultimate capacities obtained above by a factor of safety specified in 5-4/Table 1. For the Design Earthquake, the factor of safety is to be specially considered.

TABLE 1
Factor of Safety for Axially Loaded Piles

Loading Condition	*Factor of Safety*
Normal	2.0
Abnormal	1.5

9.3.2 Lateral Loads

In the evaluation of the pile's behavior under lateral loadings, the combined-load-deflection characteristics of the soil and pile, and of the pile and structure are to be taken into account. The representation of pile's lateral deflection when it is subject to lateral loads is to adequately reflect the deterioration of the lateral bearing capacity when the soil is subject to cyclic loading.

The description of the lateral load versus deflection characteristics (*p*-*y* curves) for the various soil strata is to be based on constitutive data obtained from suitable soil tests. Reference is to 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 subject to lateral loads are to be in accordance with 5-2/9.

9.5 Large Diameter Piles

The design method recommended for slender piles in 5-4/9.3 may not be applicable to large diameter piles. Due to the geometry of the pile, the failure modes of the soil may be different than those applicable for slender piles as described in 5-4/9.3. A suitable analytical method, such as the finite element analyses, may be employed to evaluate the holding capacity of large diameter piles and the adequacy of the piles to withstand in-place loads. Soil properties necessary for these analyses are to be established in consultation with the geotechnical consultant. Where the limit equilibrium method is used, different failure modes, which are affected by various factors such as the pile load and load direction, are to be considered to establish the holding capacity of a pile. Model tests, centrifuge tests, or full scale pile testing may be carried out to provide justification of the design method.

9.7 Pile Installation Analysis

Pile installation analyses including lifting, transportation and penetration are to be carried out according to the relevant requirements in API RP 2A and API RP 2SK.

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 software programs. 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* or API RP 2A if applicable.

11 Gravity Structures

11.1 General

For foundation systems consisting of gravity structures, the stability of the foundation with regard to bearing and sliding failure modes is to be investigated using the soil shear strengths determined in accordance with 5-4/5.9 and 5-4/7.3. The effects of adjacent structures and the variation of soil properties in the horizontal direction are to be considered where relevant.

Where leveling of the site is not carried out, the predicted tilt of the overall foundation is to be based on the average bottom slope of the sea floor and the tolerance of the elevation measuring device used in the site investigation program. Differential settlement is also to be calculated and the tilting of the structure caused by this settlement is to be combined with the predicted structural tilt. Any increased loading effects caused by the tilting of the structure are to be considered in the foundation stability requirements described in 5-4/11.3.

When an under-pressure or over-pressure is experienced by the sea floor under the structure, provision is to be made to prevent piping which could impair the integrity of the foundation. The influence of hydraulic and slope instability, if any, is to be determined.

Initial consolidation and secondary settlements, as well as permanent horizontal displacements, are to be calculated.

11.3 Stability

The bearing capacity and lateral resistance are to be calculated under the most unfavorable combination of loads. Possible long-term redistribution of bearing pressures under the base slab are to be considered in order to verify that the maximum edge pressures are used in the design of the perimeter of the base.

The lateral resistance of the foundation is to be investigated with respect to various potential shearing planes. Special consideration is to be given to any layers of soft soil.

Calculations for overturning moment and vertical forces induced by the passage of a wave are to include the vertical pressure distribution across the top of the foundation and along the sea floor.

The capacity of the foundation to resist a deep-seated bearing failure is to be analyzed. In lieu of a more rigorous analysis, the capacity of the foundation to resist a deep-seated bearing failure can be calculated by standard bearing capacity formulas applicable to eccentrically loaded shallow foundations, provided that

i) Uniform soil conditions are present or conservatively chosen soil properties are used to approximate a non-uniform soil condition;

ii) A trapezoidal distribution of soil pressure is a reasonable expectation,. Alternatively, slip-surface methods, covering a range of kinematically possible deep rupture surfaces can be employed in the bearing capacity calculations.

The maximum allowable shear strength of the soil is to be determined by dividing the ultimate shear strength of the soil by the minimum safety factors given below.

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 2
Factor of Safety for Allowable Shear Strength of Soil

Loading Condition	*Factor of Safety*
Normal	2.0
Abnormal	1.5

Where present, the additional effects of penetrating walls or skirts which transfer vertical and lateral loads to the soil are to be investigated as to their contribution to 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 determined and accounted for in the design of these members. The distribution of soil reactions is to be based on the results obtained in 5-4/5.7. Calculations of soil reactions are to account for any deviation from a plane surface, the load-deflection characteristics of the soil and the geometry of the base of the structure.

Where applicable, effects of local soil stiffening, nonhomogeneous soil properties, as well as the presence of boulders and other obstructions, are to be accounted for. During installation, consideration is to be given to the possibility of local contact pressures due to irregular contact between the base and the sea floor; these pressures are additive to the hydrostatic pressure.

An analysis of the penetration resistance of structural elements projecting into the sea floor below the foundation structure is to be performed. The design of the ballasting system is to reflect uncertainties associated with achieving the required penetration of the structure. Since the achievement of the required penetration of the platform and its skirts is of critical importance, the highest expected values of soil strength are to be used in the calculation of penetration.

CHAPTER 6 Marine Operations

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CHAPTER 6 Marine Operations

SECTION 1 Marine Operations

1 General

The effects that may be induced in an offshore wind turbine Support Structure during the marine operations, which are required for the installation of the structure and equipment, are to be accounted for. The emphasis of this Section is on the influence that these operations may have on the safety and integrity of the Support Structure. In this Guide, marine operations generally include the following activities as appropriate to the planned Support Structures.

i) Pile or anchor installation

ii) Final field erection and leveling

iii) Installation of Rotor-Nacelle Assembly (RNA)

For all marine operations, the Surveyor is to be satisfied that skilled supervision is being provided and that the operations are being executed satisfactorily. Refer to Chapter 1, Section 2 for the survey requirements during marine operations.

3 Documentation

The extent of documentation and analysis of marine operations is to be adequate with the size and type of Support Structure involved, the particular operation being considered, the extent of past experience with similar operations, and the severity of the expected environmental conditions.

A report on the marine operations planned to install the Support Structure is to be developed and submitted for use in association with the review of the analyses required in 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 the marine operations.

Generally, this report is to contain the following information.

i) Description of the marine operations to be performed and the procedures to be employed.

ii) Pre-installation verification procedures for the sea-bed condition and contingency procedures

iii) For operations which do not govern design of the offshore wind turbine Support Structure, a description of the engineering logic, experience or preliminary calculations supporting this conclusion.

iv) For operations which govern design of the offshore wind turbine Support Structure, the assumptions, calculations and results of the analyses required in 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 manufacturer and Operator. 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, as defined 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 lives.

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 large or unusual Support Structures, 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:

$$\frac{f \cdot S_k(f)}{\sigma_k^2} = \frac{4 f L_k / V_{hub}}{\left(1 + 6 f L_k / V_{hub}\right)^{5/3}}$$

where

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

σ_k = standard deviation of turbulent wind speed component (see A1/Table 1)

$= \left[\int_0^\infty S_k(f)df\right]^{0.5}$

L_k = integral parameter of turbulent wind speed component (see A1/Table 1)

TABLE 1
Spectral Parameters for the Kaimal model

	Wind Speed Direction		
	k=1 *(longitudinal)*	k=2 *(lateral)*	k=3 *(upward)*
Standard Deviation (σ_k)	σ_1	$0.8\sigma_1$	$0.5\sigma_1$
Integral Scale (L_k)	$8.1\Lambda_1$	$2.7\Lambda_1$	$0.66\Lambda_1$

Notes:

σ_1 = standard deviation of longitudinal turbulent wind speed

Λ_1 = scale parameter of the turbulence as specified in 3-2/11.5

Along with the Kaimal spectrum, an exponential coherence model is provided in IEC 61400-1 (2005) to account for the spatial correlation of the longitudinal wind speed:

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

where

$Coh(r, f)$ = coherence function at frequency f

f = frequency, in Hz

r = magnitude of the projection of the separation vector between the two points on to a plane normal to the average wind direction, in m (ft)

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

L_c = coherence scale parameter, in m (ft)

= $8.1\Lambda_1$, where Λ_1 is specified in 3-2/11.5

3 API Recommended Wind Spectrum and Coherence Model

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. The turbulent wind is represented by the NPD wind spectrum (also known as the Frøya wind model) and the two-point coherence function in conjunction with the logarithmic wind shear law. The wind spectrum and coherence model are adapted from API RP 2A-WSD to this Subsection. Reference is made to 3-2/3.3 for the definition of the logarithmic wind shear law.

The standard deviation of longitudinal wind speed, denoted as σ_1, is to be determined by the logarithmic wind shear law given in 3-2/3.3. In the absence of site data, the values of standard deviation of lateral and upward wind speed components are to be taken as $0.8\sigma_1$ and $0.5\sigma_1$, respectively.

3.1 Wind Spectrum

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

$$S(f) = \frac{320\phi^2\left(\frac{U_0}{10\phi}\right)^2\left(\frac{z}{10\phi}\right)^{0.45}}{\left(1+\tilde{f}^{\,n}\right)^{(5/3n)}} \qquad \tilde{f} = 172f\left(\frac{z}{10\phi}\right)^{2/3}\left(\frac{U_0}{10\phi}\right)^{-0.75}$$

where

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

f = frequency, in Hz

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

n = 0.468

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

ϕ = unit conversion factor

= 1 when using SI units (m, m/s)

= 3.28 when using US Customary units (ft, ft/s)

3.3 Spatial Coherence

The squared correlation between the spectral energy densities $S(f)$ of the longitudinal wind speed fluctuations between two points (x_j, y_j, z_j), j = 1, 2, in space is described by the two-point coherence function as follows:

$$Coh(f) = \exp\left\{-\frac{1}{U_0/\phi}\cdot\left[\sum_{i=1}^{3}A_i^2\right]^{\frac{1}{2}}\right\} \qquad A_i = \alpha_i\, f^{\,r_i}\left(\frac{\Delta_i}{10\phi}\right)^{q_i}\left(\frac{\sqrt{z_1 z_2}}{10\phi}\right)^{-p_i}$$

where the coefficients α_i, p_i, q_i, r_i and the distances Δ_i are specified in A1/Table 2, and

$Coh(f)$ = coherence function at frequency f

f = frequency, in Hz

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

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

ϕ = unit conversion factor

= 1 when using SI units (m, m/s)

= 3.28 when using US Customary units (ft, ft/s)

TABLE 2
Coefficients and Distances for the Three-dimensional Coherence Function

i	Δ_i	q_i	p_i	r_i	α_i
1	$\lvert x_2 - x_1 \rvert$	1.00	0.4	0.92	2.9
2	$\lvert y_2 - y_1 \rvert$	1.00	0.4	0.92	45.0
3	$\lvert z_2 - z_1 \rvert$	1.25	0.5	0.85	13.0

APPENDIX 2 Abbreviations and References

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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

1. ABS *Guide for Buckling and Ultimate Strength Assessment for Offshore Structures*
2. ABS *Guide for Building and Classing Mobile Offshore Units*
3. ABS *Guide for Fatigue Assessment of Offshore Structures*
4. ABS *Guide for Nondestructive Inspection of Hull Welds*
5. ABS *Rules for Building and Classing Mobile Offshore Drilling Units*
6. ABS *Rules for Building and Classing Offshore Installations*
7. ABS *Rules for Materials and Welding (Part 2)*
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9. ACI 318, "Building Code Requirements for Reinforced Concrete"
10. ACI 357R, "Guide for the Design and Construction of Fixed Offshore Concrete Structures"
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