Recommended Practice for Compliance of Large Onshore Wind Turbine Support Structures

Committee Draft (CD)

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Point of Contact (POC): Rolando E. Vega, Ph.D., P.E. rovega@absconsulting.com	THIS DOCUMENT IS STILL UNDER STUDY AND SUBJECT TO CHANGE. RECIPIENTS OF THIS DOCUMENT ARE INVITED TO SUBMIT, WITH THEIR COMMENTS, NOTIFICATION OF ANY RELEVANT PATENT RIGHTS OF WHICH THEY ARE AWARE AND TO PROVIDE SUPPORTING DOCUMENTATION.	

ASCE/AWEA

Wind Turbine Structures:

Recommended Practice for Compliance of Large Onshore Wind Turbine Support Structures

May 2011

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1 Preface

With the objective of capturing and assuring that this *Recommended Practice* document serves the need of the industry a survey and outreach team was formed to develop a survey, collect and analyze professional judgment and experience of a much larger group that included Authorities Having Jurisdiction from throughout the nation. The Summer 2010 survey was developed to have a better understanding and perspective of Authorities Having Jurisdiction with regards to: (1) permitting challenges, (2) key issues which the Project Team members may not be aware, and (3) understand the level of knowledge that exists among Authorities Having Jurisdiction with respect to wind turbine standards.

The survey received 170 responses from respondents located in 39 states. The responses were considered very helpful for capturing different regional perspectives. The survey was carried out with an online form and followed an anonymous procedure to foster objective discussion. While a larger statistical sample of the industry would have been more ideal, nevertheless, feedback obtained from this survey was valuable, discussed within the Project Team members and considered in the development of this Recommended Practice document.

The two largest groups that provided responses to the survey were Authorities Having Jurisdiction (54%) and Building Inspectors (20%) accounting for 74% of all respondents. Responses were also received from individuals identified in the other groups; specifically, Developers/Owner/Operator; Manufacturers; Design Engineers, Financier/Investors; and Others.

The developers of this Recommended Practice are considering pursuing the creation of a consensus standard with the intent that this standard would be adopted by reference into the model building codes (e.g. the International Building Code). *ASCE is an American National Standards Institute (ANSI) accredited Standards Development Organization (SDO).* The future standard would be developed in accordance with ASCE Rules for Standards Committees (the Rules) and the ASCE Standards Writing Manual based on the ANSI Essential Requirements: Due process requirements for American National Standards. The steps for developing a consensus Standard is briefly outlined in the following simplified flowchart, in accordance with the Memorandum of Understanding (MOU) between ASCE and AWEA.

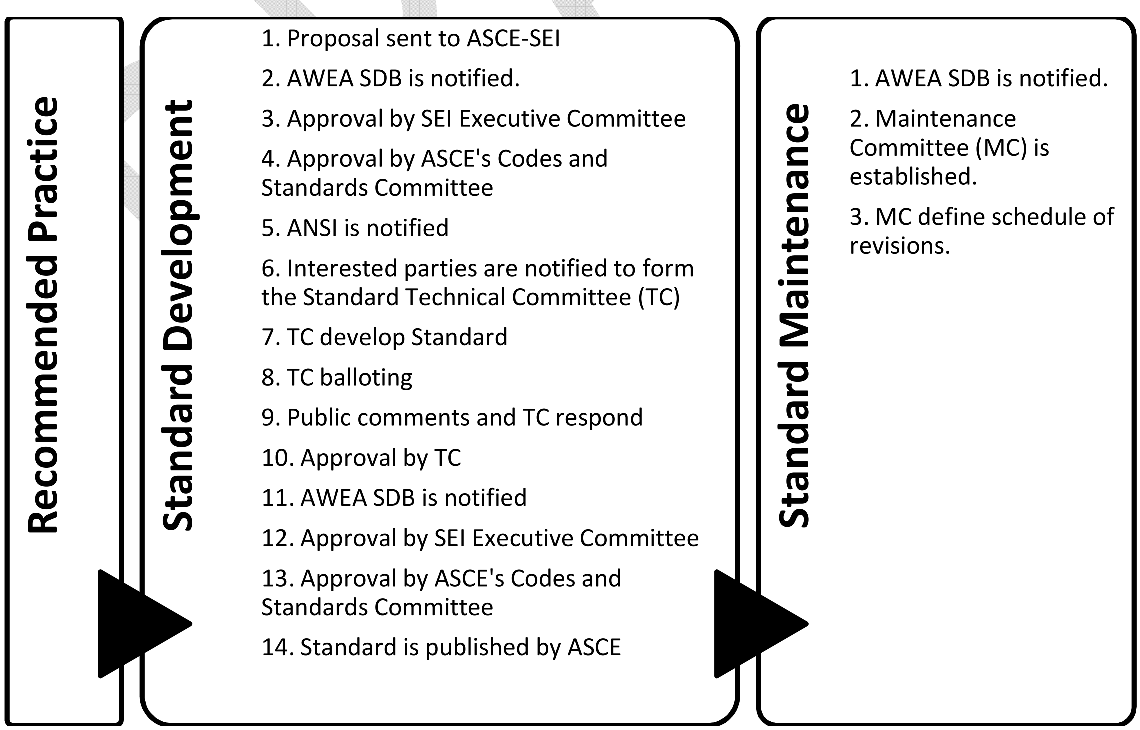


Figure 1-1: Simplified process illustration for developing a national consensus Standard on wind turbine tower and foundation structures

2 Introduction and Purpose

The *Recommended Practice for Compliance of Large Wind Turbine Support Structures* details prudent recommendations for designs and processes for use as a guide in the design and approval process in order to achieve engineering integrity of wind turbines in the U.S. The purpose of this document is to:

Enable those responsible for the permitting process to achieve consistency by clarifying the relevant and appropriate standards that have been used in the design process and should be applied when assessing structural capacity, and

Insure that wind turbine structures so permitted have an appropriate minimum level of protection against damage from hazards during the planned lifetime.

Wind turbines are constructed for the purpose of electricity generation, and are therefore elements of electrical power plants that operate in conjunction with the electrical infrastructure as a cohesive unit. They are built in diverse locations, often remote or rural, widely distributed across the United States in various legal jurisdictions. Since they are not buildings, bridges, or structures typically granted permits in many areas, the support structures for the turbines can be governed by design criteria that are not familiar to the Authorities Having Jurisdiction (AHJs) for providing construction and operating permits. There is a need to clarify the process of establishing the structural integrity of wind plants built in diverse local jurisdictions.

The American Wind Energy Association (AWEA) Standards Development Board has authorized a committee to develop documents that clearly identify typical and specific U.S. national wind turbine design recommendations that are compatible with the International Electrotechnical Commission [IEC,2005] requirements and to provide recommendation where IEC 61400-1 and U.S. practice differs. An organizing meeting of all interested parties was hosted by the National Renewable Energy Laboratory (NREL) on October 27-28, 2009. As a result of the meeting three main project teams – Structural, Offshore and Electrical – were identified to investigate the gaps and develop guidelines that address the needs of the industry. This *Recommended Practice for Compliance of Large Wind Turbine Support Structures* is the outcome of the Structures Project Team. The Offshore and Electrical project teams are publishing guiding documentation separately since there is very little overlap in permitting needs between topic areas.

International standards are already in place by which turbines are designed and which are therefore used to evaluate their structural adequacy. Almost all large wind turbines available on the market today have been certified or otherwise objectively evaluated by an international certification body through a comprehensive evaluation, testing, and manufacturing quality review process. When these turbines are introduced into the U.S. market, they must also satisfy local structural and electrical permitting requirements. Since there may be more than one standard against which a turbine is evaluated, this document also attempts to clarify the overlaps or fill the gaps between alternate standards, as well as local practice. The beneficiaries of this document are intended to be the local AHJs, by providing clarity in wind turbine structural requirements, and the developers, who must design the plant to meet local expectations, manage the construction to meet those plans, and provide appropriate supporting documentation.

This Recommended Practice is concerned with the loading and structural dynamics of on-shore wind turbine support structures. It therefore deals with subsystems that affect the response of the structural system, including control and protection mechanisms, internal electrical systems, mechanical systems, support structures (tower and foundation) and geotechnical considerations. This document provides general guidance on identification of criteria and parameters used for site evaluation, turbine selection, site-specific design, construction, commissioning and monitoring of wind plants. It deals with large, utility scale machines, which are defined in the IEC Standards as turbines with rotor swept areas larger than 200 square meters.

To be effective, this Recommended Practice document must be used together with the appropriate IEC and other international standards mentioned in this document, as well as U.S. Standards, including AISC, ACI 318, and ASCE 7. *Strength design* of steel components may be similar to or in accordance with AISC's Load and Resistance Factor Design (LRFD) [AISC, 2005]. Strength design of concrete components may be similar to or in accordance with ACI 318 [ACI 318, 2008]. A Load and Resistance Factor Design (LRFD) approach is adopted, except where serviceability limit states or other design assessments require unfactored or working stress loads.

The *Recommended Practice for Compliance of Large Onshore Wind Turbine Support Structures* was developed in conjunction with the Wind Energy Structures subcommittee of the American Society of Civil Engineers (ASCE) Structural Wind Engineering Committee. All together, the Structures Project Team consists of fifty members from Academia, Research Laboratories, Certification Bodies, Consultants and Designers, Manufacturers and Professional Societies. In addition, internal and external review panels, adding seventeen technical experts representing U.S. and Canadian Standards were engaged in the process with the objective to obtain a high level of technical accuracy in the recommendations.

3 Terms and Definitions

AISC Provisions Specification: General term to refer to the steel design provisions contained in the American Institute of Steel Construction's (AISC) standard titled ANSI/AISC 360-05 *Specification for Structural Steel Buildings* as contained in the AISC *Steel Construction Manual* [AISC, 2005].

Authority Having Jurisdiction (AHJ): The governmental agency or local building official with regulatory authority to issue structural permits for the project site.

Certification Agency: An agency that carries out type (equipment) or project (site-specific) certification of wind turbines and its components on the basis of specific IEC Standards or guidelines. In this context "certification" refers to commercial certification usually by a non-governmental third-party agency and should not be misconstrued to mean approval stamping by a Professional Engineer (PE) or approval by *AHJ* plan review, both of which are regulatory approval processes sometimes referred to as "engineering certification".

Certification Agency Rules: The design standards or guidelines that serve as the *Certification Agency's* basis of certification. Any references herein to the design provisions of particular *Certification Agency Rules* should not be construed as commercial endorsement of the associated *Certifying Agency*.

Commissioning: Quality-based process with documented confirmation that wind turbine systems are tested, balanced, operated and maintained in compliance with the owner's project requirements. Commissioning requirements for the Wind Turbine are typically defined by the Wind Turbine Manufacturer.

Complex terrain: terrain with significant variations of terrain topography failing to meet indicators shown in Section 5.4.3.6.1.

Component Class: Safety classification assigned for the design of wind turbine components based on its failure consequence, as more specifically described in Section 4.3.

Contractor – Any group procured to provide various services related to the development of Wind Turbine Generator System (WTGS).

Cut-in and Cut-out Speeds: The relative wind speed at which the wind turbine starts and stops operating for generation of power, respectively.

Developer- A group or entity responsible for forming and closing all business transactions related to the design, build and establishment of wind turbine facilities. Responsibilities generally extend from initial due diligence, land purchase, purchase power negotiation and project financing to final commissioning of the system. Responsible sub parties are hired by the developer to complete these tasks with supervision maintained by the developer.

Engineer: The designer or the engineer with design or inspection authority. Where required by the *local building code* or *AHJ*, the *Engineer* is usually a Registered Design Professional (RDP), such as a licensed Professional Engineer (PE) or and Structural Engineer (SE), or the Engineer of Record (EOR) for the permit.

Fabricated Tube: A circular steel tube created from forming flat plate into cylindrical or tapered ring segments called "cans." Cans are joined by circumferential (girth) welds to form longer tube sections. Fabricated tubes used in large utility-scale Wind Turbine Generator System (*WTGS)* towers are in almost all cases thin-shell structures with high outside diameter-to-wall thickness ratios (i.e., "D/t ratios").

Fabricator: Company responsible for fabricating tubular steel tower structure. Fabricators can build towers to Turbine Manufacturer design and specifications or Fabricators may be responsible for tower design to meet the Turbine Manufacturer loading specifications.

Field Contractor: Company or companies responsible for the installation of the Tower or Foundation elements and the required bolted, field welded or grouted connections to secure the structural system and components not pre-installed to the Tower by the Fabricator.

Foundation: Wind Turbine Generator System (*WTGS)* structural support system located below grade and responsible for transferring load to the subsoil. Geotechnical subsoil properties govern sizing of this structural support system. Details included in the foundation support system include the anchoring system from the tower to subgrade support system. Generally reinforced concrete incorporated with spread or pile footings, or other concepts are developed by a licensed Professional Registered Engineer based on the geotechnical conditions that exist.

Horizontal Axis Wind Turbine (HAWT): A wind turbine configuration with the plane of the rotor blades perpendicular to the wind direction and with the axis of rotation of the main rotor shaft lying in the horizontal plane.

Hotspot: A *stress concentration* for a welded joint. The term “hotspot” does not imply a thermal characteristic but rather denotes the appearance of high stress concentration in an FEA color contour stress plot, especially using the common color contouring convention where red color represents the highest stress intensity.

Independent Engineer: Generally an independent engineer will provide peer review or specific verification on a component or specific aspect of the system in question.

Loads Document: A report generated by the turbine manufacturer that summarizes all or primary Wind Turbine Generator System (*WTGS)* governing loads in compliance with IEC Standards or *Certification Agency Rules*, and as applicable to the design component under consideration.

Local Building Code: The building code enforced by the *AHJ* for structural permitting. In the absence of a *local building code*, the International Building Code (IBC) [IBC, 2009] may be used to represent *local building code* requirements.

Owner: Owner and developer may be or may not be synonymous. For this documents purpose we will assume the developer is working on behalf of the owner.

Project: Refers to all components and activities related to the development of wind generation. The project is generally managed by the developer.

Reference wind speed: Wind speed averaged over 10-minutes at hub height as designated for wind turbine classes.

Standard Wind Turbine Class: Wind turbine that has prescribed parameter values for reference wind speed, turbulence, temperature range, humidity etc., as indicated in Section 5.

Strength Design: A method of proportioning structural components by applying design load factors to the demand loads and reducing the component strength by applying capacity reduction factors. While the choice of design methodology rests with the *Engineer*, it is useful to observe that much of the international structural steel design practice based on the Eurocodes has long been in a strength design format. In contrast, working stress design remains in use in some structural and mechanical engineering standards in the U.S.

Stress Concentration: An area of localized high stress due to the effect of a stress riser such as a geometric discontinuity.

Support Structure: See *Tower* and *Foundation*.

Tower Fabricator: Business enterprise responsible for fabricating tower portion of the structural support system. Fabricators can build towers to Turbine Manufacturers design and specifications or Fabricators may be responsible to design tower to meet Turbine Manufacturers loading specifications.

Tower: Typically the Wind Turbine Generator System (*WTGS)* structural system mounted to the foundation and supporting the Wind Turbine. In cases where a short tube section is used as a tower top adapter or yaw adapter in connection with turbine mounting, the adapter may be classified as either part of the turbine or as part of the tower at the discretion of the *Engineer*, except that any adapter section greater than two meters in length should be considered part of the tower. Towers as classified by this definition are open to the discretion of the designer with regards to material type and geometric configuration. Generally towers supplied for WTGS applications are fabricated tube structural support systems.

Turbine Manufacturer: Business enterprise responsible for design, manufacture, delivery and sale of Wind Turbine components and in some cases the Tower. Turbine Manufacturer is responsible for establishing loads (both static and dynamic) and moments generated by the Wind Turbine transferred through the tower top adapter system.

Turbine Manufacturer: Business enterprise responsible for design, manufacture, delivery and sale of Wind Turbine components and in some cases the Tower. Turbine Manufacturer is responsible for establishing load and moments imparted by the Wind Turbine onto the Tower.

Vertical Axis Wind Turbine (VAWT): A wind turbine configuration where the main shaft's axis of rotation is vertical. This is in contrast to a Horizontal Axis Wind Wind Turbine (*HAWT)*. *VAWT* configurations such as the Darrieus type wind turbine are not within the scope of this *Recommended Practice document.*

Wind Energy Conversion System (WECS): See *Wind Turbine Generator Systems* (*WTGS*).

Wind Turbine Generator System (WTGS): An electricity-generating system consisting of a wind turbine generator elevated by mounting it on top of a *support structure* consisting of a tower and foundation. The most common example of a *WTGS* configuration addressed by this *document* is a 3-bladed upwind *HAWT*.

Wind Turbine: Consists of blades, hub, nacelle, yaw system, internal drivetrain, and electrical generator equipment.

Wind Turbine Class: Identification of wind turbine category used in design to meet the wind conditions defined in Table 5.1.

Wind Turbine Component Class: See *Component Class*.

4 Principal Elements of Permitting, Design and Quality Assurance

4.1 General

The general flow for development of wind farms can be summarized in seven steps:

1. Site evaluation
2. Wind turbine selection
3. Site-Specific Design
4. Permitting
5. Construction
6. Commissioning
7. Monitoring and Maintenance

This can be illustrated in more detail by the flowchart shown in Figure 1. A site evaluation is used to identify wind resource potential, necessary road access, transmission system availability, wind farm layout, community acceptance and other environmental considerations that may be required by permitting authorities. This evaluation should take into account both historical site-specific and non site-specific environmental data, as necessary. The environmental data required for structural design of Wind Turbine Generating Systems is discussed in Section 5. Other environmental data and analysis is often necessary for wind resource assessment, energy production estimates and to satisfy project financing requirements, which are outside the scope of this document.

As illustrated in Figure 1, Developers play the central role in collecting the necessary site information and managing the activities required to successfully navigate the project approval process. Developers, together with wind turbine and component manufacturers, financiers, designers, consultants, construction contractors and certification agencies all play active roles in driving the industry. The goal of Developers is to find, develop and optimize economical competitive solutions to produce reliable wind energy for delivery onto the electric power grid and purchase by utilities or other power purchasers. Typically, the Developer uses a multidisciplinary design team, which functionally includes wind measurement, wind turbine selection, site layout, civil, geotechnical, environmental, structural, interconnection, electrical and safety engineers. During the initial stage of project development, several wind turbine types and models are technically evaluated based on input from wind turbine suppliers and the then known site conditions. In iterative and parallel fashion, the wind project design progresses as the wind regime, interconnection, environmental permitting, and turbine selection move forward in a converging manner to an economical, and ideally optimal, wind project design. When the final wind turbine model and layout is identified by the Developer, site-specific engineering designs for constructing the wind project is prepared by the Engineer of Record and could be verified by an Independent Engineer on behalf of investors or other stakeholders. Independent third party consultants serve to provide an independent view of the project and an independent review is typically required for project financing and possibly the Developer's internal approval board.

Guidance on Wind Turbine design, manufacturing, transportation and installation is provided by the International Electrotechnical Commission IEC 61400 series of Standards and Technical Specifications. Of interest to Authorities Having Jurisdiction, the following parts of the IEC 61400 Standard are identified which establish minimum design criteria for wind turbines.

- IEC 61400-1: Wind Turbines – Design requirements
- IEC 61400-3: Design requirements for offshore wind turbines
- IEC 61400-11: Acoustic Noise Measurement Techniques
- IEC 61400-12: Power Performance Measurements of Electricity Producing Wind Turbines
- IEC 61400-13: Measurement of Mechanical Loads

- IEC 61400-21: Measurement and Assessment of Power Quality Characteristics of Grid Connected Wind Turbines
- IEC 61400-22: Conformity Testing and Certification of Wind Turbines
- IEC 61400-23: Full-scale Structural Testing of Rotor Blades

Wind turbines are generally type certified or objectively evaluated according to the Standards above and/or according to rules or guidelines developed by Certification Agencies. Type certification of wind turbines are performed by a Certification Agency. Authorities Having Jurisdiction and Developers could choose to accept type certificates using rules developed by a Certification Agency. If rules by a Certification Agency are used, documentation will indicate that type certification of the wind turbine design meets or exceeds the requirement for structural integrity and reliability achieved by IEC 61400-1.

Type certified wind turbines can be used at projects as a means for stakeholders to gain comfort that a turbine design has met certain design criteria, either to IEC or Certification Agency standards. AHJs depend on Developers to demonstrate that certain aspects of local code requirements have been met and AHJs may not be satisfied by type certification. Such authorities often will request state Registered Professional Engineer certification that the design of the system, be it the wind turbine, the foundation, or the electrical system, meets specific aspects of the local code and certification to IEC or Certification Agency rules are irrelevant in this regard (although the Engineer of Record for the local permit application may well depend on such certification in their due diligence to provide the relevant opinion). Further, Developers can select turbines based on type certification, but must still demonstrate compliance with local codes as well as prudent engineering practices (e.g., seismic conditions) and they must ultimately comply, usually with full understanding of the design of the turbine. This process allows economic flexibility when developing a project so long as the structural integrity of the turbine, tower and foundation meet local codes and prudent engineering practice. The point is that type certification is a guide to the Developer and AHJs for understanding turbine suitability given site specific conditions, and that subsequent design and/or economic adjustments must be accounted for to meet local code requirements.

Further, the reader should not confuse the focus and interests of AHJs with those of the financing parties. AHJs depend on the opinions of Registered Professional Engineers (the Engineer of Record) that are obligated to comply with state engineering regulations and local codes whereas finance parties are able to rely upon independent engineers for expert opinions but who are not necessarily Registered Professional Engineers.

Generally speaking, the Manufacturer of the selected wind turbine often secures type certificates for the Wind Turbine Generating System. The Developer or engineers on the project team (including the Engineer of Record) are responsible for ascertaining the suitability of the turbines for a site-specific wind conditions and related structural loading. Turbine and site specific suitability calculations are generally performed by the Wind Turbine Manufacturer for the Developer and these calculations can be used by the Engineer of Record for developing their application to the AHJ. An Independent Engineer may also verify the findings for the Financier. The EOR’s design of the overall Wind Turbine Generating System (and it’s design loading capacity) must meet or exceed loading conditions expected at the project site and all local building code requirements including foundation, electrical, structural, environmental and safety requirements for the site and as defined by an Authority Having Jurisdiction or Local Building Official. Specific recommendations for foundation, tower, environmental and safety requirements are presented in Sections 5 through 10 of this Recommended Practice.

The Engineer of Record is responsible for completeness of the site-specific geotechnical evaluation, compliance with zoning, land-use, set-backs, height restrictions, preparing the foundation design, while AHJs are responsible for review and approval of the submittal. A suggested compliance check-list of minimum requirements for these two parties is provided in Appendix A of this Recommended Practices. Upon satisfactory documentation, demonstration of local code compliance, and permit evaluation by the Authority Having Jurisdiction, a Construction Permit is granted. At this stage, wind turbines are generally ordered and site preparation may begin. Construction supervision and inspections of foundations, roads, buildings, etc. are to be documented by the Engineer of Record and should follow requirements provided in Section 9 of this document and the Turbine Manufacture’s installation manual.

Delivery, staging, assembly, installation and erection of the wind turbine, tower, nacelle, hub and blades are the responsibility of the Turbine Manufacturer or Construction Contractor, depending upon their contractual requirements. Assembly is to follow manufacturer specifications and instructions inclusive of mechanical completion inspections and verifications by the Turbine Manufacturer.

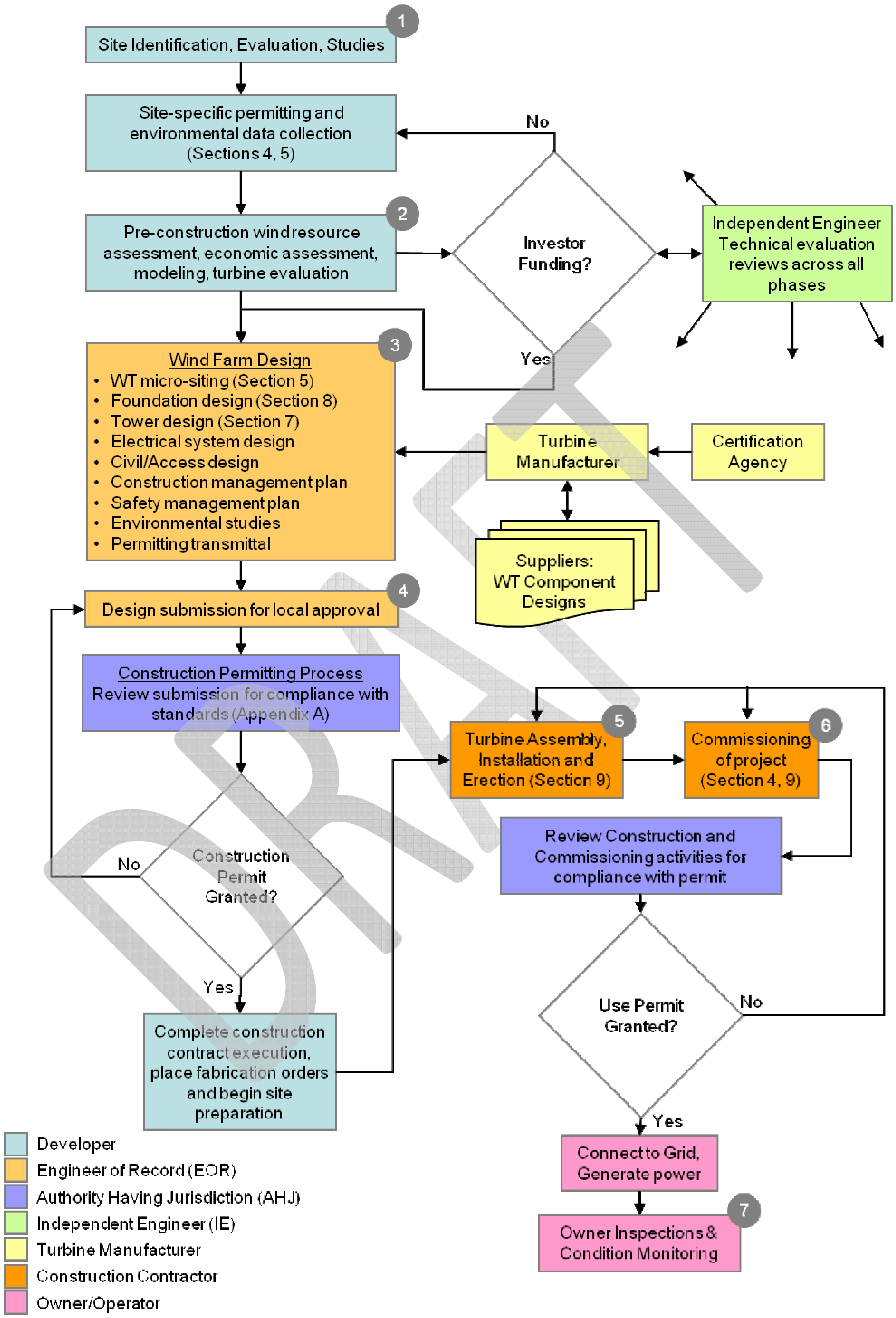
Commissioning of a wind project is typically in coordination with contractors, wind turbine manufacturer, municipalities, and transmission system operators. Upon completion of the commissioning tests, proper training of personnel for operations and maintenance of wind turbines and reports submitted to Authorities Having Jurisdiction a Use Permit is granted to cover a period equivalent to the wind turbine design lifetime. Inspections, monitoring and maintenance of wind farms are documented in the operations and maintenance manual and other proprietary records. Guidance for inspection and structural health monitoring of wind turbines is given in Section 10 of this Recommended Practices document.

4.2 Coordination of International and U.S. Standards

Since the commercial wind turbine industry evolved in Europe and because wind turbine manufacturers are part of a global market, a mix of international, European and U.S. standards in project construction documents is almost unavoidable. Recognizing that the Authority Having Jurisdiction has final authority on the interpretation of local building code requirements and that the Certification Agencies may have their own requirements, the following sections provide recommendations to assist both engineers and AHJs to reconcile international wind turbine structural design requirements with U.S. local building code requirements.

4.2.1 Conflicting Standards

The recommendations in this document should not be construed to place administrative responsibility for conflict resolution on the Engineer of Record. It is recommended that the Developer in consultation with their Engineer of Record communicates with the turbine manufacturer and the appropriate AHJ to consider strategies to accept, reject, or modify conflicting standards. Additional specific information about conflicting standards is provided in remaining sections of this document.



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633 **Figure 4-1: General wind farm project development**

4.2.2 Design Standards

Where the local building code enforced by the Authority Having Jurisdiction has regulatory authority for *WTGS support* structure design, recommendations in this document should not be construed to undermine or avoid code compliance, nor should this document be viewed to promote lesser standards than those of the local building code. However, it is recognized that IEC standards and Certification Agency rules are specialized for the purpose of WTGS support structure design. It is therefore recommended that IEC standards and Certification Agency rules serve as the primary design basis for wind WTGS structural design. The Developer and their Engineer of Record may then provide documentation to reconcile and show compliance with local building code provisions to the satisfaction of the Authority Having Jurisdiction.

Where the local building code is to serve as the primary design basis for *WTGS support* structures, it is recommended that the Developer and their Engineer of Record, in close coordination with the turbine manufacturer, ascertain whether IEC-type design load cases (DLC) would govern over the extreme wind loads, seismic load combinations, and fatigue loads developed from the local building code alone. The Engineer of Record is cautioned that the local building code's lack of specific provisions for WTGS support structures design may make it insufficient to serve alone as an appropriate design basis.

It should be recognized that from an engineering point of view (apart from regulatory concerns); the international standards utilized in the wind industry are accepted as best practice in many portions of the industrialized world, including the US. Thus, an understanding of these international standards are important for the Engineer's ability to properly design the support structure for the WTGS and the AHJs ability to rely on the standards as part of the permit application review process. The Developer in consultation with their Engineer of Record may consider the use of international design standards in lieu of US standards under the "alternative acceptance procedures" found in most standards after due consideration and the judicious use of engineering judgment and best practices. However, it should be recognized that compliance with local codes must still be demonstrated to the Authority Having Jurisdiction who has final authority to accept and rely upon alternative standards and they may require additional substantiation.

4.2.3 Quality Assurance/Quality Control

Quality Assurance for the design and permitting of wind turbine structures is achieved by the following tasks:

- Complying with industry design standards
- Site-specific design evaluation
- Project construction supervision and inspections
- Commissioning tests, operations and maintenance training
- Monitoring and Maintenance records

The following recommendations should not govern over specific provisions addressing quality assurance/quality control (QA/QC) elsewhere in this document. Conflict between US and international standards are most likely to occur between the Engineer's design and construction documents, the turbine manufacturer's specifications, and the fabricators (or contractor's) internal standards. While this Recommended Practice document makes no attempt to assign coordination responsibilities, it is recommended that coordination and conflict resolution strategies be addressed among the project team before actual conflict arises. It is therefore recommended that provisions be made for the following conditions:

– Design drawings should incorporate QA/QC requirements explicitly or by reference.

– Attempt at coordination of QA/QC (e.g., testing and inspection requirements) among the Engineer's construction documents, the fabricator's QA/QC specifications, and the turbine manufacturer's specifications.

– Creation of a baseline or default requirement that (where applicable) the local building code's inspection and testing requirements should serve as a minimum requirement. In the event of conflict with International standards, conflict may be resolved by deferring to the more stringent standard.

– In the event of disagreement on the interpretation or implementation of any aspect of the QA/QC requirements, an independent opinion should be obtained at the expense of the party promoting the lesser requirement. The independent opinion should be from a mutually agreed third party professional with expertise in the testing or inspecting methods being disputed. In some cases, the Engineer's opinion may prevail, but it is recognized that in some cases,

– QA/QC issues require detailed and specialized knowledge outside the scope of typical engineering design, such as: means and methods of fabrication; production welding processes; familiarity with the use of specific inspection equipment; etc. In these cases, the Engineer may request that a specialized welding engineer or equipment technician be consulted for an informed opinion.

– Independent Engineer may review construction quality assurance and quality control plan to assess if controls are in place to ensure compliance with design assumptions and construction specification.

– As recommended in IEC 61400-1 the quality system should comply with the requirements of ISO 9001.

4.3 Component Classifications

The integrated wind turbine system is classified according to the design parameters (i.e. reference wind speed, turbulence, temperature, humidity, etc.) in its design basis. These parameters are tabulated in IEC 61400-1, and are also shown in Table 5.1 of this Recommended Practices. This could be considered as a standard safety classification of the wind turbine system but irrespective of actual local conditions on the site. Furthermore, wind turbine components may have safety levels that depend on the consequences of failure to the global system. IEC 61400-1 tabulates values for consequence depending on the component in consideration. In addition, safety factors for loading depending on its type; and material safety factors depending on the failure mechanism are presented in Section 5. These safety factors in IEC 61400-1 can, to some degree, be compared to the importance, load and strength reduction factors, respectively, in the US standards. The values for these factors according to IEC 61400-1 are given in Section 5. In this section it is relevant to distinguish between the three given component consequence groups.

Component Class 1 (CC1) – load-bearing (structural) component that its failure would <u>not</u> result in major failure of the wind turbine (fail-safe structural components).

Component Class 2 (CC2) – load-bearing (structural) component that its failure would result in major failure of the wind turbine (non fail-safe structural components).

Component Class 3 (CC3) – mechanical component that is connected to the main structure and is used as part of the turbine protection system (non fail-safe mechanical components).

4.4 Occupancy Category

Where it is necessary to determine the Occupancy Category as defined in ASCE 7, WTGS may be classified as Occupancy Category II structures, resulting in normal design importance factors. The "power generating stations" item under Occupancy Category III, resulting in higher design importance factors, typically applies to conventional power plants capable of generating continuous power. In contrast, wind farms cannot generate continuous power nor should a WTGS be relied upon for continuous or on-demand power for essential or emergency response facilities and other Occupancy Category III or IV facilities. In general, higher importance factors would result in design conservatism, to which this Recommended Practice document have no objection. Proximity or association of the WTGS installation with other Occupancy Category III or IV structures may require that the WTGS installation match the higher classification by default. Where it is proposed to use a lower Occupancy Category classification than that of the associated facility or project, it is recommended that the Engineer seek approval from Authority Having Jurisdiction to do so.

5 External conditions and loads

5.1 General

As stated in IEC 61400-1, the appropriate level of safety and reliability, environmental, electrical and soil parameters should be taken into account and explicitely stated in the design documentation.

The following sections present a general picture of the external conditions considered in the design of a wind turbine according to IEC 61400-1 and provide design checks for compliance with specific external conditions covered in ASCE 7-05 for the U.S.

The primary external condition affecting structural integrity of wind turbines are the wind conditions and these are separated in two types: (1) normal conditions and (2) extreme conditions. Normal conditions generally concern recurrent structural loading during normal operation of a wind turbine, and extreme conditions represent rare external design conditions defined as having a 1-year and 50-year recurrence periods.

The wind conditions defined in this section are generally concerned with a constant 10-minute flow combined, in many cases, with either a varying deterministic gust profile or with turbulence. Specific turbulence characteristics for longitudinal, lateral and upward directions, turbulence scale parameter, power spectral densities and wind field coherence are given in IEC 61400-1. These turbulence characteristics are commonly considered in the design of wind turbines. When siting a wind turbine in a given place compliance of such turbulence conditions on site should be verified by either compliying with the terrain exposure characteristics on the site or with site-specific data as may be required for complex terrain.

5.2 Wind turbine classes

Wind turbines are designed and generally type-certified according to the following turbine classes. Turbines are basically categorized according to an extreme reference wind speed and turbulence as shown in Table 5.5-1. In this industry it is common practice to use reference wind speeds averaged over 10-minutes at wind turbine hub-height. When other external conditions such as temperature range, humidity, electrical power network conditions, etc., are within prescribed values shown in IEC 61400 Section 6.2, Classes I, II and III are also known as standard wind turbine classes. These are intended to cover most applications. However, these do not give precise representation of any specific site; do not cover offshore conditions, thunderstorm events, low level jets or tropical storms such as hurricanes. Site specific conditions need to be verified as discussed later in Section 5.4.7 and 5.4.8. In applications where the standard wind turbine class is not suitable, the wind turbine will be classified as Special (S) to cover those specific conditions.

Table 5.5-1: Basic parameters for wind turbine classes

Wind turbine class	I	II	III	S
V_{ref} (m/s)	50	42.5	37.5	Values specified by the designer
$V_{ref\text{-}ASCE7}$ (mph)	**See Section 5.3.4, 5.4.7 and 5.4.8 for conversion to ASCE (basic) wind speed**			
A I_{ref} (-)	0.16 (see Section 5.3.6)			
B I_{ref} (-)	0.14 (see Section 5.3.6)			
C I_{ref} (-)	0.12 (see Section 5.3.6)			

In Table 5.5-1, parameter values refer to hub height except for $V_{ref\text{-}ASCE7}$ which is meant as a conversion to the common basic wind speed in ASCE 7 and as defined below.

V_{ref} is the reference wind speed averaged over 10 minutes.

$V_{ref\text{-}ASCE7}$ the ASCE 7 reference (basic) wind speed averaged over 3-seconds at 10 meters over flat open terrain need to be converted to 10-minute average at hub height. Conversion of these wind speeds to different hub heights can be accomplished with the equation shown in Section 5.3.4.

A	category for higher turbulence (correspond to Exposure B in ASCE 7)
B	category for medium turbulence (correspond to Exposure C in ASCE 7)
C	category for lower turbulence (correspond to Exposure C in ASCE 7)
I_{ref}	is the expected value of the turbulence intensity at 15 m/s.

5.3 External conditions required for assessment

In addition to the basic parameter values of Table 5.5-1, standard wind turbine classes are designed for normal wind conditions, extreme wind conditions and other environmental conditions including temperature, air density, etc. The standard wind turbines classes do not account for detailed characteristics of thunderstorm events, tropical storms or earthquakes. However, understanding that these events are common in many jurisdictions in the U.S. recommendations to address basic thunderstorms, hurricane effects and eartquakes are provided here.

ASCE 7 is based on a neutrally stable atmospheric boundary layer model for strong winds. It may also be applied to hurricane winds. Its primary purpose is to provide wind load recommendations for the design of conventional structures and buildings. However, characterization of non-neutral, thermally driven winds is not addressed in ASCE 7. IEC provides detailed information about normal and extreme wind conditions as presented in the following sections. The extreme wind speed model (EWM) of IEC can be compared to ASCE 7 provisions.

The following models are adopted from IEC 61400-1 with the observations below:

5.3.1 Normal Wind speed probability distribution

A Weibull wind speed probability density function is recommended as a function of the reference 10-minute wind speed for the standard wind turbine class at hub height (Table 1). This is important to characterize wind speed frequency and fatigue load spectrum produced by loads between cut-in and cut-out wind speeds.

5.3.2 Normal wind profile model (NWP)

The 10-minute average wind speed as a function of height is defined with the power law with respect to the hub height and with exponent of 0.2 for the standard wind turbine class. The shear of IEC model is more conservative (i.e. the change of wind speed between lower and upper heights of the blade is greater).

Therefore Equation 5-1 from IEC is recommended as the most conservative average wind speed velocity profile for open terrain.

$$V(z) = \left(\frac{z}{z_{hub}}\right)^{\alpha} \cdot V_{hub} \qquad \text{(Eq 5-1)}$$

α = 0.2 for normal wind conditions

Appendix C shows that ASCE 7 velocity profile and IEC 61400 velocity profile match well for open terrain with little or no obstructions. Terrains with Exposure D (lower turbulence) should use velocity profile from ASCE 7 modified for exposure as given by Eq C6-1.

5.3.3 Normal turbulence model (NTM)

A linear expression is given for wind speed standard deviation as a function of wind speed at hub height. When divided by the wind speed at hub height to obtain turbulence intensity as function of wind speed, an exponential-like function shows the decreasing turbulence intensity with increasing wind speed. ASCE 7 provides an expression for turbulence intensity but this is not a function of average wind speed which is needed for the assessment of design load cases in section 5.3.43

5.3.4 Extreme wind speed model (EWM)

The conversion from 10-minute average to 3-second gust in IEC is nearly identical to ASCE 7 (i.e. Durst's averaging time correction of 1.52/1.1 ≈ 1.4 based on ASCE 7 commentary).

For extreme wind speeds use Equation 5-1 with power law exponent,

α = 0.11 for extreme gust profiles should be used.

Appendix C shows that ASCE 7 velocity profile and IEC 61400 velocity profile match well for open terrain with little or no obstructions. Terrains with Exposure D (lower turbulence) should use velocity profile from ASCE 7 modified for exposure as given by Eq C6-1.

The exponent in IEC 61400-1 is 0.11 and in ASCE 7 is 0.11 (i.e. 1/9.5 for open terrain). Therefore the extreme gust wind speed profile model in IEC 61400-1 and ASCE 7 are identical for open terrain. It is in this provision where IEC 61400-1 indicates the consideration of ±15 degree of yaw misalignment to allow for short-term deviations from the 10-minute average wind direction. This provision of potential yaw misalignment should be verified for hurricane and extreme thunderstorm regions by a wind engineer in consultation with the manufacturer. In absence of site-specific advice, a yaw misalignment of ±15, 45, 90 and 180 degrees during parked conditions is a recommended evaluation to consider the possibility that a fast moving thunderstorm could change directions faster than the yaw mechanism can react. See Design Load Case (DLC) 6.1 in Table 5.5-2.

Thunderstorm events have a different wind speed model than extreme synoptic or hurricane events. Wind speeds in thunderstorms are produced by a number of mechanical and thermal mechanisms and are generally defined by a gust front or a nearby downdraft that produce a nose-like velocity profile (i.e. not increasing wind speed indefinitely with height). ISO 4354 (2009) recommends the use of a velocity profile that could increase the 10-meter wind speed by as much as 15-20% near hub heights in the range of 50-100m. It is recommended that the site specific investigation account for such wind speed velocity profile where the extreme wind event at 10-meters is extrapolated with this profile.

Wind speed profiles for tropical cyclones (hurricanes) have produce a wide scatter of results in research. The basic agreement found in ISO 4354 with regards to extreme wind velocity profiles is that the power law (or logarithmic law) profiles described in meteorological literature applies near the ground and up to 500 meters.

5.3.5 Extreme operating gust (EOG)

When analyzing the wind turbine in the time domain for specific manoeuvres it is necessary to consider the extreme gust as a function of time. The extreme operating gust is considered in fault conditions during power production, start-up and shut-down. Section 6.3.2.2 of the IEC 61400-1 (2005) document presents a trigonometric expression for wind speed at hub height as a function of time. In the absence of well-documented extreme operating gusts for hurricanes and thunderstorms at hub-height, IEC 61400-1 extreme operating gust will remain as the standard baseline evaluation.

5.3.6 Extreme turbulence model (ETM)

During the operational state of a wind turbine, in addition to normal turbulence as a function of average wind speed (Section 5.3.3), the high or extreme wind natural turbulence needs to be considered. Section 6.3.2.2 of the IEC 61400-1 (2005) presents an expression for extreme turbulence for use within cut-in and cut-out speeds of the turbine. For wind speeds beyond cut-out speed ASCE 7 provide an equation to estimate turbulence intensity as a function of height and this should be verified for compliance with U.S. Standards.

Equation 6-5 of ASCE 7-05 shows how turbulence intensity can be calculated as a function of height. ASCE 7 describes the turbulence intensity for rough terrain (ASCE 7 Exposure B = IEC 61400-1 exposure A), Open Terrain with scattered obstructions (ASCE 7 Exposure C = IEC 61400-1 exposure B) and flat terrain or facing shallow water bodies (ASCE 7 Exposure D = IEC 61400-1 exposure C). For example, for a hub height of 80 meters, ASCE 7 turbulence intensity for high, medium and low turbulence levels are 0.21, 0.14, and 0.11, respectively. For flat open terrain the turbulence characteristics are exactly the same in IEC 61400-1 and ASCE 7. For rougher and smoother (water-like) exposures it is recommended site-specific verifications be undertaken to account for the differences in turbulence, especially for the rougher terrain as IEC 61400-1 may give less conservative designs.

5.3.7 Extreme direction change (EDC)

Large direction changes are not uncommon, particularly at low wind speeds (turbine start-up). IEC 61400-1 specifies in Section 6.3.2.4 a transient direction change in such instances with a duration of 6 seconds. Furthermore, IEC 61400-1 specifies maximum extreme direction changes that decrease with increasing wind speed. It specifies a maximum EDC of 30 degrees in 6 seconds for extreme wind speeds which according to IEC 61400-1 definitions, might include thunderstorms during operational wind turbine state). Unless indicated otherwise by the Authority Having Jurisdiction, it should be a reasonable undertaking to follow IEC 61400-1 EDC.

5.3.8 Extreme coherent gust with direction change (ECD)

During power production without faulty conditions, a time domain analysis is necessary to verify structural integrity to indentify dynamic response under extreme gusts across the rotor area. For this reason, and similar to the extreme operating gust and extreme direction change, a transient wind speed and direction change function is specified in IEC 61400-1 for these input conditions.

5.3.9 Extreme wind shear (EWS)

The normal wind profile only accounts for a uniform positive shear in the power law expression (monotonic increase in wind speed with height). During power production (between cut-in and cut-out) many other meteorological conditions arise where the atmospheric shear changes dramatically in time, vertically and horizontally. IEC 61400-1 provides an expression to account for these vertical and horizontal shears which impose large moments about the rotor axis in a transient fashion.

5.3.10 Other environmental conditions

In addition to wind conditions many other variables can impact the design of a wind turbine. The following list of parameters needs to be verified with the standard turbine class values in IEC 61400-1 or in the design documentation. There are normal and survival temperature ranges to be considered. For example, normal temperatures of relevance to structural design will have minimum range of -20C to +50C.

- Temperature
- Humidity
- Air density
- Solar radiation
- Rain, hail, snow and ice
- Chemically active substances
- Mechanically active substances
- Salinity
- Lightning

5.4 Loads and load calculations

5.4.1 General

In general, loading should be in accordance with IEC 61400-1 [IEC, 2006] or Certification Agency Rules. Under no circumstance should these loadings be allowed to produce a design safety level that would be less than that required by the *local building code.* In the absence of a *local building code*, the IBC and ASCE 7 standard may be used to represent *local building code* requirements. In addition to *local building code* prescribed loads and load combinations this document recommends “best practice” load combinations that consider the combination of wind and seismic loading that is unique to *WTGS* support structures.

In practice, wind turbine manufacturers may provide a *Loads Document* created in accordance with IEC 61400-1 Standards or *Certification Agency Rules*. The loads therein are typically generated using highly specialized (and often proprietary) software capable of dynamic load simulation. To show compliance with the *local building code*, it is recommended that the tower *Engineer* compare the *Loads Document* extreme wind design load to show that it meets or exceeds the *local building code's* extreme wind load. The *Engineer* should also evaluate the earthquake plus operational load combinations appropriate for the project site.

5.4.2 Wind turbine modelling

Companies involved in the analysis of wind turbine modelling; whether as Consultants, Manufacturers or Designers, should consider the entire generation system which include a variety of mechanisms that work in synchronization. Among the mechanisms that need to be considered are:

- Control functions
- Protection functions
- Braking system
- Errors of fitting
- Hydraulic or pneumatic systems
- Main gearbox
- Yaw system
- Pitch system
- Protection function mechanical brakes

5.4.2.1 Loading mechanisms

The design process should be able to handle loading from a number of sources as applicable to the site-specific conditions, and allow for the different load safety factors involved in the process. An good aero-servo-elastic code should consider the following:

- Gravitational and inertial loads
- Aerodynamic loads
- Actuation loads
- Other loads (wake effect, impact, ice loads)

5.4.2.2 Local Coordinate System

The following figure shows the coordinate system used to define forces and moments in the tower and the foundation of the structure. Mainly, the z-direction is vertical upward along wind turbine tower; the x-direction is pointing downwind parallel to wind turbine drive train axis (i.e. turbine main shaft axis); and the y-direction is perpendicular to drive train axis.

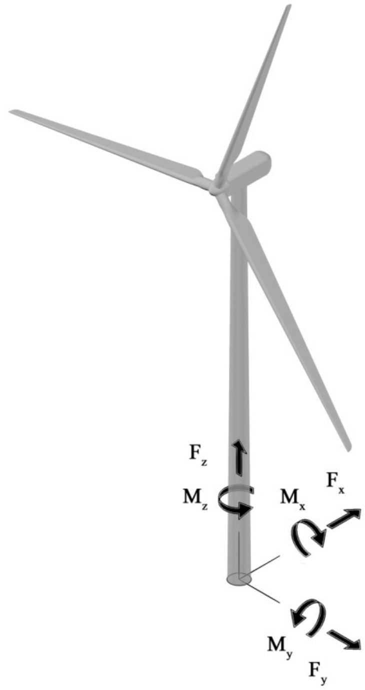

Figure 5.5-1: Wind tower and foundation coordinate system for forces and moments

5.4.3 Design situations and loads cases

5.4.3.1 General

When designing a wind turbine a minimum number of design situations need to be considered to cover worst loading conditions as recorded and used for the design of its components. These loading conditions can occur during start-up, power production, shut down, still or idling, transport, assembly, maintenance and repair phases of construction and operation. These conditions must also consideroccurrence of faults (control or protection system failure or loss of electrical network) during operation and still or idling conditions. The minimum number of design situations and load cases are covered in thorough detail in IEC 61400-1 (IEC, 2005) Section 7.4. These design load cases from IEC 61400-1 are shown in Table 5.5-2 for reference purposes. Other design load cases should be considered, if relevant to the structural integrity of the specific wind turbine design. For seismic or hurricane-prone regions refer to Sections 5.4.4 of these *Recommended Practice*, respectively.

Table 5.5-2: Design load cases (IEC 61400-1, 2005 with English Units)

Design Situation	DLC	Wind conditions	Other conditions	Type of analysis	Partial Safety Factor
1) Power production	1.1	NTM $V_{in} < V_{hub}\ V_{out}$	For extrapolation of extreme events	U	N
	1.2	NTM $V_{in} < V_{hub}\ V_{out}$		F	*
	1.3	ETM $V_{in} < V_{hub}\ V_{out}$		U	N
	1.4	ECD $V_{hub} = V_r \pm 4.5$mph and = V_r		U	N
	1.5	EWS $V_{in} < V_{hub}\ V_{out}$		U	N
2) Power production plus occurrence of fault	2.1	NTM $V_{in} < V_{hub}\ V_{out}$	Control system fault or loss of electrical network	U	N
	2.2	NTM $V_{in} < V_{hub}\ V_{out}$	Protection system or preceding internal	U	A

			electrical fault		
	2.3	EOG $V_{hub} = V_r \pm 4.5$mph and = V_{out}	External or internal electrical fault including loss of electrical network	U	A
	2.4	NTM $V_{in} < V_{hub}$ V_{out}	Control, protection, or electrical system faults including loss of electrical network	F	*
3) Start up	3.1	NWP $V_{in} < V_{hub}$ V_{out}		F	*
	3.2	EOG $V_{hub} = V_{in}$ $V_{hub} = V_r \pm 4.5$mph and = V_{out}		U	N
	3.3	EDC $V_{hub} = V_{in}$ $V_{hub} = V_r \pm 4.5$mph and = V_{out}		U	N
4) Normal shut down	4.1	NWP $V_{in} < V_{hub}$ V_{out}		F	*
	4.2	EOG $V_{hub} = V_r \pm 4.5$mph and = V_{out}		U	N
5) Emergency shut down	5.1	NTM $V_{hub} = V_r \pm 4.5$mph and = V_{out}		U	N
6) Parked (standing still or idling)	6.1	EWM 50-year recurrence period		U	N
	6.2	EWM 50-year recurrence period	Loss of electrical network connection	U	A
	6.3	EWM 1-year recurrence period	Extreme yaw misalignment	U	N
	6.4	NTM $V_{hub} < 0.7$ V_{ref}		F	*
7) Parked and fault conditions	7.1	EWM 1-year recurrence period		U	A
8) Transport, assembly, maintenance and repair	8.1	NTM V_{maint} to be stated by the manufacturer		U	T
	8.2	EWM 1-year recurrence period		U	A

Abbreviations used in Table 5.5-2:	
DLC	Design load case
ECD	Extreme coherent gust with direction change
EDC	Extreme direction change
EOG	Extreme operating gust
EWM	Extreme wind speed
EWS	Extreme wind shear
NTM	Normal turbulence model
ETM	Extreme turbulence model
NWP	Normal wind profile model
$V_r \pm 4.5$mph	Sensitivity to all wind speeds in the range should be analyzed
F	Fatigue
U	Ultimate strength
N	Normal
A	Abnormal
T	Transport and erection
*	Partial safety for fatigue

5.4.3.2 Safety factors

Safety factors for the design of wind turbines are defined somewhat similar to U.S. Standards. In the U.S. Standards, there are three safety factors: facility importance factor, material strength reduction factor and load factor. In the design of wind turbines there are three safety factors: component consequence factor, material safety factor and loading safety factor.

As discussed in Section 4.4, the category of wind power facilities can be considered such that an Importance Factor of 1.0 applies for their overall design, however, depending on the consequence of failure of a given component a consequence factor will apply. In most cases applicable of this Recommended Practice a Consequence Class 2 applies as failure of the support structure may lead to the failure of a major part of the wind turbine. In these cases, except for fatigue design the safety level due to consequence has a factor of 1.0 which is the same as the Importance Factor of 1.0.

Material partial safety factors or its reciprocal, strength reduction factors, should be carefully evaluated in each case. For the design of steel towers it is thought that material safety factors are comparable in IEC 61400-1 and those in AISC, but for the design of foundations IEC 61400-1 is thought to be less conservative in some cases. As more research becomes available a more specific recommendation will be given in future revisions of this Recommended Practice or in the development of a Standard.

Loading safety factors in IEC 61400-1 are in general more comprehensive as it includes many wind turbine load cases. However, it should be noted that for the design of facilities in the U.S. a loading safety factor on the extreme 50-year wind conditions of 1.5 applies.

5.4.3.3 Limit state analysis

Ultimate limit state analyses make use of partial safety factors to account for the uncertainties and variability in loads and materials, the uncertainties in the analysis methods and the importance of structural components with respect to the consequences of failure. These partial safety factors relate characteristic loads and material strengths to their design values. The partial safety factors that ensure safe design values are defined in the following equations:

$$\boldsymbol{F_d = \gamma_f F_k} \qquad \text{(Eq 5-2)}$$

where

F_d is the design value for the aggregated internal load or load response
γ_f is the partial safety factor for loads and
F_k is the characteristic value for the load.

$$\boldsymbol{f_d = \frac{1}{\gamma_m} f_k} \qquad \text{(Eq 5-3)}$$

where

f_d is the design values for materials
γ_m is the partial safety factor for materials; and
f_k is the characteristic value of material properties.

The partial safety factors for loads take account of possible unfavorable deviations of the loads from their characteristic values and uncertainties in the loading model. The partial safety factors for materials used in these *Recommended Practice* take account of possible unfavorable deviations of the strength of materials relative to their characteristic value, inaccurate assessment of the resistance of sections or load carrying capacity of parts of the structure, uncertainties in geometric characteristics, conversion factors, and the relation between the material properties in the structure and those measured by tests on control specimens.

The general limit state condition that relates partial safety factors with loads or load cases, including those in Table 5.5-2, and material strength properties along with the consequences of failure is the following:

$$\gamma_f F_k \leq \frac{1}{\gamma_m \gamma_n} f_k \qquad \text{(Eq 5-4)}$$

where γ_n is the partial safety factor for the consequences of failure. This limit state equation is applicable to different analysis types, including ultimate strength, fatigue, stability, and critical deflections. A summary of the partial safety factors and their associated analysis types is given in Table 5.5-3:

Table 5.5-3: Analysis types and partial safety factors for limit state load and resistance verifications

Analysis Type	γ_f	γ_m [a]	γ_n	
Ultimate Strength Analysis	Table 5.5-4	≥ 1.1[b]	CC1	0.9
		1.2 for global buckling of curved shells such as tubular towers and blades	CC2	1.0
		1.3 for rupture from exceeding tensile or compression strength	CC3	1.3
Fatigue Analysis	1.0	≥ 1.7 for components with large coefficient of variation for fatigue strength, i.e., 15% to 20%	CC1	1.0
		≥ 1.5 provided that the SN curve is based on 50% survival probability and coefficient of variation < 15%	CC2	1.15
		≥ 1.1 for welded and structural steel provided the SN curve is based on 97.7% survival probability		
		≥ 0.9 for welded and structural steel provided the SN curve is based on 97.7% survival probability with periodic inspection to detect critical crack development	CC3	1.3
Stability Analysis	Table 5.5-4	≥ 1.1[b]	CC1	1.0
		1.2 for global buckling or curved shells such as tubular towers and blades	CC2	1.0
		1.3 for rupture from exceeding tensile or compression strength	CC3	1.3
Critical Deflection Analysis	Table 5.5-4	1.1 except when the elastic properties have been determined by full scale tests in which case it may be reduced to 1.0	CC1	1.0
			CC2	1.0
			CC3	1.3

[a] Partial safety factors for materials where recognized design codes are available need to be combined and they cannot be less than those specified in Table 5.5-3 for the respective analysis type.
[b] Applies to characteristic material properties of 95 % survival probability with 95 % confidence limit. This value applies to components with ductile behavior.

Table 5.5-4: Partial safety factors for loads γ_f

Unfavorable loads			Favorable loads
Type of design situation (See Table 5.5-2)			All design situations
Normal (N)	Abnormal (A)	Transport and Erection (T)	All design situations
1.35*	1.1	1.5	0.9

* A partial safety factor for loading of *1.5* should be considered where hurricane or thunderstorms control the design. That is according to ASCE 7 load safety factor equal to 1.6 times the directionality factor of 0.95 for round structures (including tubular towers).

Regarding special safety factors relative to Table 5.5-3, lower partial safety factors for loads may be used where the magnitudes of loads have been established by measurement or by analysis confirmed by measurement to a higher than normal degree of confidence. The values of all partial safety factors used should be properly stated in the design documentation.

5.4.3.4 Power production

In this design situation, a wind turbine is running and connected to the electric load. Unlike conventional building structures, *WTGS* are subject to loads generated from the dynamic operation of the wind turbine machinery. While the IEC 61400-1 design load cases consider many operational design load cases, the *local building code* load combinations do not explicitly consider these. For this reason, when the *local building code* is used for design basis of *WTGS* support structure, operational loads should be considered.

As *WTGS* are subject to high-cycle fatigue loading, fatigue loading should be considered in the design of *WTGS* towers. The fatigue loading should represent operational conditions considering a variety of possible wind speed regimes and other operational events.

5.4.3.5 Power production plus occurrence of fault or loss of electrical network connection

This design situation involves a transient event triggered by a fault or the loss of electrical network connection while the turbine is producing power and is significant for wind turbine loading. Some *WTGS* are subject to operational or abnormal operational fault loads considered in the IEC 61400-1 standards that exceed the *local building code* extreme wind loads. Further, the *local building code* most likely has no method or provisions for assessing these potentially governing design loads. For this reason, compliance with the local building alone may not necessarily produce an adequate tower design or produce a design that meets wind industry standards. It is imperative that the Design *Engineer* coordinate with the turbine manufacturer to determine whether there are loading conditions that potentially exceed the *local building code* extreme wind or earthquake loads. Fatigue loading should also be considered.

5.4.3.6 Other relevant conditions

Other relevant conditions included in the design load cases (DLCs) in Table 5.5-2 are:

- Start-up
- Normal shut-down
- Emergency shut down
- Parked (standstill or idling)
- Parked plus fault conditions
- Transport, assembly, maintenance and repair

5.4.3.7 Special load case verifications (as applicable)

5.4.3.7.1 Assessment of the topographical complexity of the site

If terrain topography does not meet the following indicators of Table 5-5 given by IEC 61400-1, then a complex terrain asessment needs to be performed. For gentle changes in terrain ASCE 7-05 could be used to determine the increase or decrease in wind speed with respect to the position of the wind turbine and the ridge or top of the topographic feature. For cliffs or abrupt changes in terrain more advanced models are needed, including use of wind tunnels or computational fluid dynamics as advised by a wind engineer.

Table 5-5: Terrain complexity indicators

Distance range from wind turbine	Maximum slope of fitted plane	Maximum terrain variation from a disc with radius 1.3 z_{hub} fitted to the terrain
< 5 z_{hub}	< 10 degrees	< 0.3 z_{hub}
< 10 z_{hub}		< 0.6 z_{hub}
< 20 z_{hub}		< 1.2 z_{hub}

5.4.3.7.2 Assessment of wake effects from neighbouring wind turbines

Wake effects produced by neighbouring wind turbines during power production should be considered including single or multiple wakes from upwind machines. This should include effects of spacing between machines for all operational wind speeds and wind directions.

Wake effects produce a reduction in the speed of mean flow and an increase in the turbulence intensity. The increase in loading can be considered by the use of an effective turbulence intensity that account for discrete and turbulent wake effects.

Recommendations for the calculation of effective turbulence intensity, and the calculation of wake effects from neighbouring wind turbines are given in Annex D of IEC 61400-1.

5.4.3.7.3 Assessment of other environmental conditions

The following environmental conditions should be compared to to the assumptions made in the design of a wind turbine:

- Normal and extreme temperature ranges
- Icing, hail and snow
- Humidity
- Lightning
- Solar radiation
- Checmically active substances
- Salinity

5.4.4 Seismic loading and design criteria

5.4.4.1 General

This section presents criteria for the design of *WTGS* subject to earthquake ground motions. At sites with increased seismic hazard *WTGS* have a reasonable likelihood of being in an operational state during an earthquake and may also be subjected to simultaneous earthquake and emergency stop loads if a shutdown is triggered by the earthquake. In addition to the earthquake load combinations in the *local building code*, the *WTGS* support structure design should consider load combinations that include operational loads plus earthquake loads. Seismic design criteria and load combinations may be in accordance with *Certification Agency Rules*, as recommended in this section, or as justified by rational engineering.

5.4.4.2 Seismic ground motion values

Seismic ground motion values should be determined per ASCE 7-05 Section 11.4 or the site-specific ground motion procedures set forth in ASCE 7-05 Chapter 21 (as permitted in ASCE 7-05 Section 11.4.7). Spectral response acceleration parameters should be based on 5% damped values per standard practice.

5.4.4.3 Geologic hazards and geotechnical investigation

Consideration of seismic forces should be included in the foundation design for areas with historical earthquake activity. Evaluation of earthquake effects should be performed in accordance with local building codes, or IEC 61400-1 requirements. Guidance in seismic loading evaluation is provided in the ASCE 7 standard which, by reference, is part of the building codes in the majority of US jurisdictions. Earthquake loads should be evaluated based on ground motion parameters and response spectra required by the applicable local building code. Where the IBC is the applicable building code, seismic design parameters should be provided in all geotechnical reports, regardless of whether the project is located in a seismically active region or not. Building codes in most jurisdictions are based on, or default to, the IBC.

Where IBC code governs, geotechnical evaluation of earthquake effects should include, but not be limited to, the following:

- Ground shaking
- Liquefaction
- Slope instability
- Surface fault rupture
- Seismically induced settlement/cyclic densification
- Lateral spreading
- Cyclic mobility
- Soil strength loss

Liquefaction susceptibility analysis may be performed using state-of-the-art analytical and empirical procedures based on SPT, CPT or shear wave velocity of the subsurface materials. Appropriate mitigation should be provided for foundations located in areas where analysis indicates susceptibility to earthquake effects noted above. The selected method should be at the Geotechnical Engineer or foundation Designers discretion, but within the wind industry and project location standard of care.

Where a project is located near active faults, turbines should be located with adequate setbacks from the fault zone. The characteristics of the fault including type, seismic setting, subsurface conditions, ground motion attenuation, and maximum earthquake magnitude that can be generated from the fault should be considered. Ground shaking should be accounted for appropriately using analysis procedures provided in ASCE 7 or in accordance with local building code requirements. The seismic design category for each project should be assigned and the seismic loading analysis procedure selected accordingly with consideration of site specific spectral accelerations, structural period, and dynamic characteristics.

Where relatively loose unsaturated cohesionless soils are present at a given site, the effect of ground shaking from a design level earthquake should be taken into account. Potential settlement due to cyclic densification of the site soils should be evaluated.

5.4.4.4 Performance Objectives

Building code seismic design requirements do not ensure that structures will be operational after a design level earthquake. Similarly, a *WTGS* support structure designed using these *Recommended Practices* may be damaged during an earthquake beyond a level that is economically repairable, or turbine components may be rendered inoperable by the earthquake induced shaking. This level of performance may not present a significant risk to human life due to the relative frequency that *WTGS* are occupied and the remote location of many wind farms, but the potential economic losses may represent an unacceptable risk to the wind farm owner. Enhanced performance objectives, such as operational performance after a design level earthquake, may be established to meet specific owner requirements. However, a clear distinction should be made between the required minimum performance objectives of the *local building code* (life-safety) versus that of any supplementary, project-specific contractual agreement.

A performance factor (similar to an importance factor of 1.5 for essential facilities) may be established to improve expected behaviour during and after an earthquake. A major consideration in establishing this performance factor is coordinating with the *WTGS* manufacture to establish acceleration thresholds for turbine components that will ensure operational performance (e.g. operational performance if nacelle acceleration during an earthquake is limited to some maximum value). Using advanced analysis techniques and the established turbine component thresholds the Engineer can evaluate support structure options that will achieve these improved performance goals.

5.4.4.5 Seismic Load Combinations

WTGS have a reasonable likelihood of being in an operational state during an earthquake and may also be subjected to simultaneous earthquake and turbine emergency stop loads if a shutdown is triggered by the earthquake. ASCE 7-05 seismic load combinations do not include consideration of concurrent earthquake and turbine operational loads. It is of critical importance to recognize that seismic plus operational loads may in some cases govern tower and foundation design. Therefore, for engineering “best practices” it is suggested to evaluate seismic plus operational load combinations, regardless of the absence of a codified requirement. The following “best practices” load combinations including seismic plus operational loads are recommended and should be considered in addition to ASCE 7-05 prescribed load combinations:

Seismic Load Combination:

$$U = (1.2 + 0.2S_{DS})D + 0.75(\rho Q_E + 1.0M) \quad (1)$$

$$U = (0.9 - 0.2S_{DS})D + 0.75(\rho Q_E + 1.0M) \quad (2)$$

where,

M = operational loading equal to the greater of: 1) loads during normal power production at the rated wind speed; or 2) characteristic loads calculated for an emergency stop at rated wind speed.

ρ = 1.0, redundancy factor (for nonbuilding structures not similar to buildings ρ = 1.0 per ASCE 7-05 Chapter 12.3.4.1).

All other variables are as defined in ASCE 7-05 Sections 2, 11, 12, and 15.

Note that for the load combinations, the minimum specified value of the seismic response coefficient, C_s, per ASCE 7-05 Equations 15.4-3 and 15.4-4 should apply, in lieu of values from Equations 15.4-1 and 15.4.2.

It is suggested that operational and earthquake loads be combined as an absolute sum with a load factor of 0.75. Use of a load factor of 0.75 on both the earthquake and operational loads is similar to a square root sum of squares type combination and is supported by results of response history analysis of wind turbines (Prowell 2010). Alternatively, other methods may be used to combine operational and earthquake loads provided that they are justified by rational engineering analysis.

5.4.4.6 Analysis Procedures

Any analysis procedure (equivalent lateral force, modal response spectrum analysis, or response history analysis) permitted by the *local building code* is acceptable for use with these *Recommended Practices*. Refer to the *local building code* or ASCE 7 for specific requirements for each analysis procedure.

If the equivalent lateral force procedure is used the vertical distribution of seismic forces should be calculated based on the procedure given in ASCE 7-05 Chapter 12.8.3 with the following modifications. The effective seismic weight of the nacelle and rotor should be assumed to be located at the turbine's center of gravity, and the effective seismic weight of the tower structure (including ladders, platforms, railings, etc.) should be distributed to nodes located at tower can joints. Seismic forces should also be assumed to act at these locations.

ASCE 7-05 Table 15.4-2 presents response modification factors, R, for various nonbuilding structures not similar to buildings, but does not explicitly include *WTGS* support structures. These *Recommended Practices* suggest the use of R = 1.5 unless a larger value is justified by rational engineering analysis that is reviewed and accepted by the Engineer of Record and building official.

In practice, often only the peak seismic loads and peak operational loads are available. As a result, the proposed combination method for operational and seismic loads may be an overly conservative approach, especially considering that the respective peak loads do not occur at the same instant of time and in the same loading direction. Seismic response history analysis, considering time varying earthquake ground acceleration and operational or emergency stop loads, can be used to more accurately predict response and reduce potential design conservatism. Response history analysis results should be evaluated with respect to the performance objectives outlined in Section 5.4.4.4. Seismic response history analysis procedures should conform to the requirements of ASCE 7-05 Chapter 16. It is suggested that any such analysis be conducted with analysis software capable of simulating both structural response and global turbine dynamics, including aerodynamic interaction.

5.4.5 Assessment of soil conditions

Each wind project shall have a site-specific geotechnical study to determine geotechnical parameters for the proposed foundations and associated load transfer mechanisms. The Geotechnical Engineer shall conduct the work with the degree of skill and care exercised by other Geotechnical Firms working in the wind energy industry with consideration of geotechnical standards in the region that the services are performed.

5.4.5.1 Geotechnical Document Review

A review of available geotechnical and geologic documentation should be conducted as part of the geotechnical investigation scope of work. Typical documentation review includes the following, as applicable:

- Historical and current aerial photographs,
- Published regional geologic maps,
- Soil survey reports,
- Groundwater hydrology data and maps,
- Landslide mapping,
- Solution cavity (sinkhole) mapping
- Mine subsidence mapping
- Seismic hazard mapping,
- Slope stability analysis, if determined necessary
- Other applicable geotechnical and geologic documentation.

5.4.5.2 Geotechnical Exploration

Geotechnical exploration for each turbine site should consist of at least one exploration point per foundation, or more as necessary to characterize soil and bedrock conditions within the foundation influence zone. As a general guide, subsurface exploration points should be located within the footprint of the proposed turbine foundation. Geotechnical exploration should be of a sufficient depth in order to determine subsurface characteristics within the foundation influence zone. For shallow foundations, exploration should also be a minimum depth at least equal to the foundation base width. If refusal is encountered at shallower depths in high strength soils, not all explorations need always be continued to the full depth, at the discretion of the Geotechnical Engineer in consultation with the designer. For deep foundations, exploration should be at least the maximum anticipated foundation depth, plus an additional 20 percent.

In-situ exploration methods, including cone penetration testing, flat plate dilatometer testing, vane shear testing, and other in-situ methods should be supplemented by an appropriate amount of soil borings in order to correlate in-situ data with laboratory testing.

5.4.5.3 Geophysical Testing

Geophysical testing, including seismic velocity testing, local gravity, and other methods, often proves useful to assist in determining soil properties for turbine foundation design. Geophysical investigations should be carried out by a licensed professional with specific experience in the geophysical method to be used. Geophysical methods should only be used to supplement the subsurface exploration program and never be used as the only means of geotechnical exploration.

Seismic testing, including downhole seismic, Seismic CPT, and surface methods should be conducted at a representative number of sites in order to determine shear and compression wave velocity of the subsurface materials. The shear and compression wave velocities can then be used to determine dynamic shear modulus and be input into dynamic analyses of the foundation.

Other geophysical testing methods may be used to investigate presence of groundwater, subsurface voids, locate geologic discontinuities, interpolate between exploration points, and many other aspects of wind farm development.

5.4.5.4 Groundwater Considerations

Effects of groundwater should be accounted for in the turbine foundation design, which may require relatively long term monitoring of groundwater levels at the specific foundation locations during the geotechnical investigation. Long term groundwater levels should be incorporated into stability, bearing capacity and other pertinent foundation design evaluations.

The geotechnical engineer shall determine the design groundwater level, which should take into account seasonal fluctuations as well as long-term groundwater levels. The foundation design shall account for any effects of buoyancy resulting from the design groundwater level. The design groundwater level may or may not vary across the site.

5.4.5.5 Geotechnical Laboratory Testing

Laboratory testing should be conducted on samples from soil borings gathered during the subsurface exploration program to determine engineering properties for design of the proposed foundations. Laboratory testing should be sufficient to characterize all soil types and layers that may have an impact on the foundation design. The following laboratory tests should be included in the soils laboratory testing program, as applicable:

- Moisture content and unit weight
- Plasticity indices
- Grain size analysis
- Shear strength (unconfined, triaxial, direct shear, vane shear, etc.)
- Consolidation
- Compaction characteristics (maximum unit weight, optimum moisture content, etc.)
- Corrosivity characteristics (Sulfate, chloride, pH, resistivity, etc.)Other geotechnical laboratory testing as appropriate.

5.4.6 Assessment of Frequency Separation

To avoid resonance, *WTGS* should be designed with sufficient separation between system natural frequencies and turbine operational frequencies. The calculation of *WTGS* system natural frequency should account for the mass and stiffness properties of the turbine, tower, and foundation. The operational frequencies should include the turbine primary rotor operational frequency (i.e., the “1xp” frequency) and the blade-pass frequency (e.g., the “3xp” frequency, for a 3-blade turbine). Any other significant loading known to act as a harmonic forcing function should also be considered.

Many turbine manufacturers install test or prototype turbines in the field for verification of various operational parameters including frequency separation. Prototype verification data may be used in lieu of analytical frequency calculations.

Where applicable, frequency separation should comply with the *Certification Agency Rules*. In the absence of such criteria, the following may be applied, which closely approximate the frequency separation criteria in GL(2003):

1. Approximate fundamental period methods such as that in ASCE 7 Section 12.8.2.1 should not be used for determining frequency separation. The system natural frequencies should consider the stiffness and mass properties of the entire *WTGS* system, which includes the turbine, tower, and foundation. The calculated nominal system frequencies should be varied by 5% to account for tolerances in design assumptions and calculations. Alternatively, to account for variations, tThe system natural frequencies should may be represented by an upper bound (i.e., stiff/rigid) estimate and a lower bound (i.e., soft/flexible) estimate. The upper bound frequency may be calculated on the basis of a minimum system mass estimate with an assumption of an infinitely rigid foundation. The lower bound frequency estimate may assume the maximum system mass estimate with the maximum turbine mass moment of inertia value and the minimum permissible foundation rotational stiffness value.

2. The system natural frequencies should have a minimum 5% separation from the operational frequencies. To account for the recommended 5% tolerance in calculated values, the total minimum separation would be 10%, i.e., 5% separation plus 5% tolerance. Separation by 5% or less may be considered a high risk resonance condition and may not be acceptable, but vibration mitigation strategies described in some *Certification Agency Rules* may be considered. General wind industry experience has shown that 15% minimum frequency separation is quite adequate and preferred, since almost no significant instances of tower resonance have been observed or reported at that amount of separation margin.The calculated system natural frequencies should preferably have 15% minimum separation from the turbine operational frequencies. As a minimum a 10% separation should be used. Separation between 5% to 10% may indicate risk of resonance, and engineering discretion is advised. Separation by 5% or less may be considered a high high-risk resonance condition and may not be acceptable, but vibration mitigation strategies described in some *Certification Agency Rules* may be considered.

5.4.7 Assessment of structural integrity by reference to wind data

When the 50-year extreme wind climate accounting for any local effects (topography, wake effect from neighboring turbines, exposure/turbulence) is found to comply with the reference wind speeds of Table 5.5-1, outside of hurricane prone regions, the turbine is suitable for the site. Under this circumstance, the support structure may be designed with loading data from the pertinent standard wind turbine class, usually provided by the manufacturer. Specific verification for turbulence characteristics and consideration of normal wind speed probability density function in the design of the wind turbine is provided in IEC 61400-1 (2005) Section 11.9. Reference to wind data and determination of 50-year recurrence periods, as established by ASCE 7 (2005), should comply with the following:

Outside hurricane-prone regions:

Wind speeds from ASCE 7 wind map (Figure 6.1 of same Standard) can be used as reference value to be compared with Table 5.5-1 standard wind turbine classes.

Any less stringent wind speeds than those defined in ASCE 7 wind map, and from regional climatic data should only be used when:

1. Approved extreme-value analysis has been employed
2. Length of record, sampling error, averaging time, anemometer height, data quality, and terrain exposure of anemometer have been taken into account.

In hurricane-prone regions:

The use of regional wind speed data obtained from anemometers should not be permitted.

Any less stringent wind speeds than those defined in ASCE 7 wind map, should account for the following:

1. Approved simulation and statistical analyses.
2. Design wind speeds resulting from the study should not be less than the 500-year return period divided by $\sqrt{1.5}$.

5.4.8 Assessment of structural integrity by load calculation with reference to site-specific conditions

Structural integrity can be demonstrated by comparison of loads and deflections for the site-specific conditions with those used in the design basis of the standard wind turbine class. In regards to verification of loads for extreme conditions, while any rational method should be permitted, the following minimum verification is recommended. The extreme wind associated with the IEC 61400-1 site class rating should be compared to the ASCE 7 design wind. The comparison should consider all applicable ASCE 7 wind parameters including the wind exposure, wind profile, and topographic factor.

The ASCE 7 extreme wind is defined as a 3-second gust, 50-year wind at 10 meter height. Further, the ASCE wind load factor of 1.6 when adjusted by the directionality factor for round structures is approximately 1.5. In comparison, IEC 61400-1 Design Load Case (DLC) 6.1 is a 3-second, 50-year wind at hub height with a design load factor of 1.35. For this reason, it is recommended that IEC 61400-1 DLC 6.1 be used to verify compliance with *local building code* extreme wind loads.

In addition, IEC 61400-1 state specific verification for ultimate and fatigue effects produced by wake effects from neighbouring turbines for design load cases 1.1 and 1.2.

6 Materials

For material specifications, see the materials section for the specific structure type.

7 Tower Support Structure

This section, herein referred to as these *Recommended Practices*, addresses the structural design of towers in *WTGS* and apply to the support structure types defined in this *Guideline*. This section applies to steel *fabricated tube* towers of circular cross section.

At this time, *local building codes* are not sufficiently specialized for *WTGS* tower design and as such may not necessarily be appropriate to serve as a basis for tower design. Nevertheless, in the current regulatory process of structural permitting, a *WTGS* project may be subject to the design requirements of the *local building code*. For the purpose of complete tower design, full compliance with *Certification Agency Rules* is recommended as a design basis, followed by design validation against *local building code* requirements. Where a proven tower design already exists in service, e.g., already installed and in production operation overseas, and was designed in accordance with *Certification Agency Rules*, a limited design assessment or plan review to show compliance with the *local building code* may be considered sufficient. However, the required extent of the design assessment with respect to satisfying the *local building code* can only be determined by the *AHJ*. Any less stringent standard or provision from *Certification Agency Rules* should not be used to undercut or violate the *local building code* requirements, unless the *Engineer* has been granted permission to do so by waiver from the *AHJ* or another agency with the appropriate authority.

Similarly, any less stringent standard from the *local building code* should not undercut or violate the *Certification Agency Rules*. If the situation is unavoidable, the *Engineer* should report the conflicting condition to the owner or client. In the event of conflicting standards, the *local building code* should prevail; however, it is the owner's or client's responsibility to determine the effect of *local building code* compliance on the conditions of commercial certification.

7.1 Materials

The following lists represent materials currently in common use in the wind industry. This list is not exhaustive, nor should it be construed to prohibit unlisted but otherwise suitable materials.

Tower Shell:

- ASTM A36: Carbon structural steel.
- ASTM A572: High-strength structural steel.
- ASTM A709: Structural steel for bridges.
- EN 10025-2 S235: Structural steel
- EN 10025-2 S355: Structural steel

Tower Splice Flanges and Base Plates:

- Forged Flanges: ASTM A694
- Forged Flanges: EN 10025-3 S355
- Cut or formed from plate: See Tower Shell steels listed above.

High Strength Bolts:

- ASTM A325: Structural bolts
- ASTM A490: Structural bolts alloy steel
- EN 14399-4: High-strength structural bolting assemblies for preloading - Part 4: System HV - Hexagon bolt and nut assemblies (M12 to M36) together with EN 14399-6: High-strength structural bolting assemblies for preloading - Part 6: Plain chamfered washers.

- DASt Guideline 021: Hot dipped galvanized bolt assemblies (M39 to M64)

7.2 Strength Design

In this *Guideline*, *strength design* is the recommended design methodology. This maintains compatibility with International practice and follows recommended practice in the US.

In general, design strength may be calculated in accordance with the following:

- Certification Agency Rules, if applicable
- AISC Provisions [AISC, 2005] for steel design.
- Eurocode 3: Design of steel structures
  - EN 1993-1-1 : General rules and rules for buildings
  - EN 1993-1-6 : Strength and stability of shell structures
- DIN 18800 [DIN-18800], 2008-11 for steel design.

In general, fatigue strength should be calculated in accordance with the following:

- Eurocode 3: EN 1993-1-9 [EC3-9, 2005] for fatigue design of steel structures.
- For fatigue design of concrete structures refer to the Foundation Section.

Design strength of *fabricated tube* towers may be calculated in accordance with *Certification Agency Rules,* or design standards that address thin shell tubes such as EN 1993-1-6 or DIN 18800. Where design strength is to be calculated in accordance with US design standards, the following issues should be considered:

1. Use of AISC provisions for round tubes and pipes may be problematic for fabricated tube tower design on several accounts: the design provisions have explicitly stated limits of applicability to only HSS sections; the maximum D/t slenderness ratio considered in the AISC provisions is limited to a maximum of $0.45E/F_y$, which is routinely exceeded in *WTGS* towers; the available axial compressive strength based on flexural buckling is not easily determined for a tapered fabricated tube with varying cross sectional wall thickness (i.e., the so-called "stepped column" problem), which cascades into further complicating the calculation of the flexure and axial force interaction.
2. Past US and European tower design practices have drawn from established design standards for other similar structures. In particular, having close structural similarities to fabricated tube towers is the steel stack (chimney-type) structure, whose design in the US is governed by the ASME steel stack standard [ASME, 2006]. The aforementioned European steel thin shell design provisions and the ASME stack standard share a similar design approach. Where local buckling governs, as in virtually 100% of all practical fabricated tube towers, flexural and axial stress are combined into a single normal stress. That normal stress is compared against the local buckling capacity of the thin shell at the considered location. The European standards have a *strength design* format whereas the ASME standard uses a working stress design format (i.e., allowable stress design).

For the reasons stated, the following provisions are assembled from US standards and may be used to satisfy the requirement that design strength be calculated in accordance with US standards. AISC's LRFD format is used. The procedure in [Troitsky, 1990] is used but with the upper transition slenderness limit modified according to [ASME, 2006]. Shear and torsion interaction are according to [Galambos, 1998]. The resulting equations are identical to many of the AISC provisions but not subject to the AISC limits of applicability to HSS only.

7.2.1 Compressive Strength

The tower shell subject to compression should meet the following condition:

$f_u \leq \phi_c F_n$ (Eq. 7-1)

Where,

$f_u = P_u/A + M_u/S$ (Eq. 7-2)

P_u = Design vertical force, usually equal to $-F_z$. Note that the sign should be consistent with the sign of the flexural stress component.

A = Area of tower cross section.

M_u = Design moment, usually M_{xy}.

S = Elastic section modulus of tower cross section.

The design compressive strength, $\phi_c F_n$, should be determined as follows:

The nominal compressive strength, F_n , should be the lowest value obtained according to the limit states of yielding, flexural buckling, or local buckling.

$F_n = F_{cr}$ (Eq. 7-3)

$\phi_c = 0.90$

Slenderness parameters:

$\lambda = D/t$ where D is the outside diameter of the tower shell and t is the shell thickness.

$\lambda_1 = 0.11E/F_y$

$\lambda_2 = 0.357E/F_y$

λ_{MAX} = No maximum value is specified, but it is rare in most practical tube towers to find slenderness values in excess of 330.

For $\lambda \leq \lambda_1$

F_{cr} should be the smaller of the following:

$F_{cr} = F_y$ (Eq. 7-4)

F_{cr} due to flexural buckling calculated in accordance with “stepped column” procedures such as those in [Barnes, 1979] or [Newmark. 1943].

For $\lambda_1 < \lambda \leq \lambda_2$

$F_{cr} = QF_y$ (Eq. 7-5)

where $Q = 0.038E/[F_y\,(D/t)] + 2/3$ (Eq. 7-6)

For $\lambda_2 > \lambda <= \lambda_{MAX}$

$F_{cr} = 8000/(D/t)$

(Eq. 7-7)

7.2.2 Shear Strength

The tower shell subject to transverse shear should meet the following condition:

$f_{vu} \leq \phi_v F_{vn}$ (Eq. 7-8)

Where,

$f_{vu} = V_u/A_v$ (Eq. 7-9)

V_u = Design shear force, usually equal to F_{xy}.

A_v = Shear area equal to half the gross area, $A_g/2$.

Equations for the critical shear buckling stress of cylindrical shells can be derived from Section 14.3.3 of [Galambos, 1998]. The results are identical to AISC Equations G6-2a and G6-2b. Therefore, this *Guideline* recommends the use of those AISC equations for the calculation of shear strength of round fabricated tube towers.

The design shear strength, $\phi_v F_{vn}$, should be determined as follows:

The nominal shear strength, F_{vn} , should be the lowest value obtained according to the limit states of shear yielding and shear buckling.

$F_{vn} = F_{cr}$ (Eq. 7-10)

$\phi_v = 0.90$

F_{cr} for circular fabricated tubes should be determined as the larger of the following:

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \qquad \text{(Eq. 7-11)}$$

And

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \qquad \text{(Eq. 7-12)}$$

but F_{cr} should not exceed $F_y/\sqrt{3}$,

Where,

L = Length between stiffened tower cross sections, e.g., tower section length between splice flanges.
D = Tower wall outside diameter.
t = Nominal tower wall thickness.

Note that F_{cr} as defined above acts on a cross sectional shear area equal to half the gross area, $A_g/2$.

7.2.3 Torsional Strength

The tower shell subject to torsion should meet the following condition:

$f_{Tu} \leq \phi_T F_{Tn}$ (Eq. 7-13)

Where,

$f_{Tu} = T_u/C$ (Eq. 7-14)
T_u = Design torsional moment, usually equal to M_z.
C = Torsional section modulus, J/r, where J is the polar moment of inertia and r is the distance to the center of rotation.

Equations for the critical torsional buckling stress of cylindrical shells can be derived from Section 14.3.3 of [Galambos, 1998]. The results are identical to AISC Equations H3-2a and H3-2b. Therefore, this *Guideline* recommends the use of those AISC equations for the calculation of torsional strength of round fabricated tube towers.

The design torsional strength, $\phi_T F_{Tn}$, should be determined as follows:

The nominal torsional strength, F_{Tn} , should be the lowest value obtained according to the limit states of torsional yielding and torsional buckling.

$F_{Tn} = F_{cr}$ (Eq. 7-15)

$\phi_T = 0.90$

F_{cr} for circular fabricated tubes should be determined as the larger of the following:

$$F_{cr} = \frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}}$$

(Eq. 7-16)

And

$$F_{cr} = \frac{0.60E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}}$$

(Eq. 7-17)

but F_{cr} should not exceed $F_y/\sqrt{3}$,

Where,

L = Length between stiffened tower cross sections, e.g., tower section length between splice flanges.
D = Tower wall outside diameter.
t = Nominal tower wall thickness.

7.2.4 Combined Torsion, Flexure, Shear and/or Axial Force

The tower shell subject to combined forces should meet the design condition as for the force or moment acting alone and also the following conditions:

$f_{vu} /(\phi_v F_{vn}) + f_{Tu} /(\phi_T F_{Tn}) <= 1.0$ (Eq. 7-18)

and

for $f_{Tu}/(\phi_T F_{Tn}) \leq 0.20$

$f_u/(\phi_c F_n) <= 1.0$ (Eq. 7-19)

for $f_{Tu}/(\phi_T F_{Tn}) > 0.20$

$[\, f_u/(\phi_c F_n)]^2 + [\, f_{vu} /(\phi_v F_{vn}) + f_{Tu} /(\phi_T F_{Tn})\,]^2 <= 1.0$ (Eq. 7-20)

The 20% torsional shear trigger for interaction with normal stresses is used similar to that of the AISC provisions. However, in contrast to AISC, these *Guidelines* have elected to use elliptical interaction of the normal and shear forces for reasons including: historical performance of *WTGS fabricated tube* towers indicates no obvious need for conservatism with respect to shear stresses; some degree of consistency with EN 1993-1-6; and there may be conservatism in the approach that considers the normal stress as the sum of the axial compression stress and a dominant flexural stress against design resistance derived from only axial compression.

7.3 Fatigue strength

Investigating fatigue strength for a WTGS involves the consideration of complex loading combinations due to the responses of the turbine and the supporting structure to the varying nature of wind. There are no industry accepted simplified methods for determining fatigue loading appropriate for large wind turbine (LWT) support structures. Conservative assumptions regarding fatigue are too costly considering the number of structures often involved with WTGS and the fact that fatigue loading is often the governing loading consideration for many components of the supporting structure.

Most WTGS's are modeled as a complete system using complex simulators coupled with nonlinear structural and fluid dynamic models. The simulators model the entire WTGS, from the flexible blades of the rotor to the support structure itself. In many cases, testing is used to verify or supplement the results of a simulation.

The IEC 61400-1 Standard outlines specific operating and loading conditions for investigating fatigue strength. The simulators generate turbulent winds that allow the determination of the load ranges and number of cycles that can be expected over the life of the WTGS. The results are specific to a given WTGS. Changes to the support structure, such as height, stiffness or attaching appurtenances may significantly affect the results of a simulation. The IEC 61400-1 standard recommends that a minimum 20-year life be considered for WTGS.

Most fatigue investigations for the supporting structure involve analyzing the data generated from a simulation and comparing the results to published fatigue life curves (S-N curves) for the critical components of the supporting structure. The loading sequences generated by the simulation generate a cyclic loading history. The number of cycles associated with each load range is determined using established methods of analyzing data such as the Rainflow Counting procedure outlined in ASTM E1049, "Practices for Cycle Counting in Fatigue Analysis". The results are summarized in a load range spectrum which provides the frequency of occurrence for all load ranges. The load range spectrum is generally provided in the loads document for the WTGS.

Fatigue investigations are based on elastic analysis methods using unfactored operating loading conditions. The design focus is to consider the number of load applications, the stress ranges and the types and locations of critical structural components in order to prevent the initiation of a fatigue crack or the propagation of a fatigue crack from a defect, discontinuity or stress concentration. Fracture mechanics may be used to establish stress levels to minimize the potential for brittle failure considering loading rate, temperature, material toughness and the expected discontinuities from fatigue loading or from the fabrication process.

The AISC and AWS standards contain special fabricating, inspection and installation requirements for fatigue sensitive structures that should be followed for WTGS supporting structures. Turbine manufacturers may also supplement these requirements with more stringent requirements.

7.3.1 S-N Curves

S-N curves are most often presented as log-log plots with the allowable stress range on the vertical axis and the allowable number of cycles on the horizontal axis (refer to Figure 7-1). The use of a log-log plot allows for the simplified representation of an S-N curve as a series of straight lines with different slopes. The variable "m" is commonly used to designate the slope of a line on an S-N curve.

Most S-N curves account for stress concentrations and are intended to be compared to calculated nominal component stresses without the application of additional stress concentration factors. S-N curves for structural steel components generally transit into a fatigue threshold or cut-off zone at a low stress range that may be repeated for an indefinite (infinite) number of cycles without initiating fatigue damage.

EN Standard 1993-1-9 (EN) and the AISC Specification for Structural Steel Buildings (AISC) publish a family of S-N curves for various component categories. The basis of the S-N curves are identical; however, the EN standard has established additional S-N curves that fall between the AISC S-N curves that allows for a finer categorization of components with respect to their notch sensitivity (refer to Figure 7-1).

The EN and AISC S-N curves are based on research by Keating and Fisher (1986) and are based on identical confidence and probability levels. The EN family of curves include additional size affect factors and considerations for non-welded and stress relieved welded components. Although some detail requirements vary between the EN and AISC component categories, the basic concepts of both standards are the same.

The EN S-N curves are designated using Design Categories (DC) that are equal to the allowable stress range at 2 million cycles for each category. The AISC S-N curves for direct stress are designated using letters starting with A for the least notch sensitive category to E for the more severe notch sensitive category. Category F is used to define the S-N curve for shear stress.

The EN and AISC S-N curves indicate a fatigue threshold stress range below which an indefinite number of constant amplitude cycles may be applied without initiating fatigue damage. The EN S-N curves also indicate a cut-off stress range at 100 million cycles that may be used when damage summation methods are used to investigate fatigue strength. Summation methods are useful when a stress range spectrum is available that provides the magnitude and frequency of each stress range expected. Stress levels below the cut-off may be ignored in the summation; however, some turbine manufacturers require that the cut-off limit be conservatively ignored resulting in all stress ranges contributing to the summation.

One significant difference between the EN and AISC S-N curves is the magnitude of the stress range and the number of cycles at the constant amplitude fatigue threshold limit (refer to Figure 7-1). The EN fatigue threshold limit is set equal to the allowable stress range at 5 million cycles for all design categories. The number of cycles at the AISC S-N fatigue threshold limit varies between 2 to 22 million cycles depending on the notch severity of the component category.

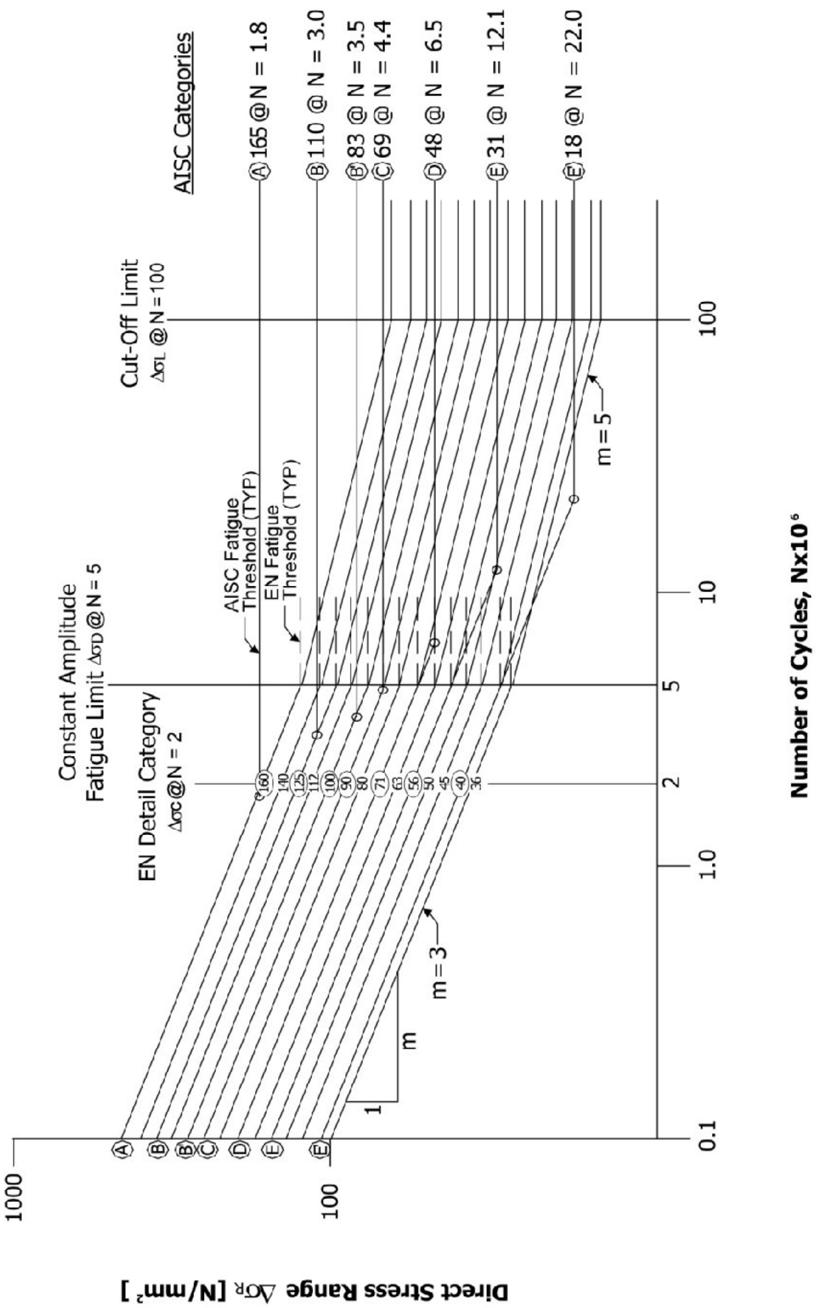

Figure 7.1: EN and AISC fatigue strength curves

7.3.2 Strength Resistance Factors

The AISC allowable stress ranges are intended to be used with a strength resistance factor equal to 1.0 based on the reliability level used to establish the S-N curves and the special material and inspection requirements for fatigue sensitive structures. The EN standard provides a refinement by presenting partial safety factors (reciprocal of strength resistance factor) that may be applied as a safety factor to the S-N curve allowable stress ranges.

The EN partial safety factors for fatigue strength vary between 1.00 and 1.35 depending on the consequence of failure and the level of inspection. The equivalent AISC strength resistance factors would vary between 0.74 and 1.00. As mentioned above, AISC assumes that special base metal considerations are used for components governed by fatigue and that inspections during fabrication and throughout the life of the structure are performed. Under these conditions, the EN partial safety factor would equal 1.00 resulting in equivalent allowable stress ranges compared to AISC.

The IEC 61400-1 standard provides a further refinement compared to the EN Standard. IEC 61400-1 separates the partial safety factor for fatigue strength into two components; one based on the importance of a component in the WTGS and one based on material considerations. A 1.15 partial safety factor for importance (component class 2) applies to the components of the supporting structure. The partial safety factor for material varies between 0.9 and 1.1 depending on the level of inspection for fatigue damage. The combined partial safety factor for fatigue using these values varies between 1.04 and 1.27. The equivalent AISC fatigue strength resistance factor varies between 0.79 and 0.97. When the reduced partial safety factor for material based on inspection is used, the IEC 61400-1 combined partial safety factor would result in approximately equivalent allowable stress ranges compared to AISC.

The AWS D1.1 Structural Welding Code prior to 1996 had a provision to use a 0.80 strength resistance factor for non-redundant structures (Ref AWS C-2.14.4). This provision was discontinued due to the reliability basis of the specified S-N curves and due to the special material and fabrication inspection requirements for fatigue sensitive structures.

7.3.3 WTGS Simulations

The dynamic simulation of a WTGS produces the data used to determine the expected load ranges at various locations on the supporting structure and the number of cycles associated with each load range. The load ranges are often reported with specific mean load values. The mean load value does not significantly affect the fatigue life for most structural components; therefore, the number of cycles for a load range is commonly determined by summing all the cycles for the load range, regardless of their mean load level.

In order to determine fatigue damage, nominal stress ranges in critical components from the applied load ranges should be determined using an elastic analysis. The stress ranges in each critical component along with their associated allowable number of cycles are used to determine demand-to-capacity ratios (DCR). A DCR value of unity or less indicates adequate fatigue strength.

The DCR for a component is generally determined by a damage summation method or by assessing stress ranges. The Palmgren-Miner's Summation method (Miner's Rule) is the most commonly used damage summation method. Assessment of stress ranges is accomplished by calculating a damage equivalent load (DEL) which is then used to determine the constant amplitude stress ranges in the supporting structure. Each method is described in the following paragraphs.

7.3.4 Miner's Rule Summation

A Miner's Rule summation involves totaling the calculated accumulative affects of fatigue damage from the load cycles determined from a WTGS simulation.

Miner's Rule summation assumes that the fatigue damage from each load cycle is accumulative and that the incremental fatigue damage from a specific load cycle equals the ratio of the number of cycles that the load cycle occurs to the total number of cycles allowed. The summation of the ratios for all load cycles becomes the fatigue demand-to-capacity ratio (DCR).

Minor's Rule summation may be expressed using the following equation:

$$DCR = \sum \frac{n_i}{N_i}$$

where:

i = load range number

n_i = number of cycles for load range i

N_i = allowable number of cycles for load range i

7.3.5 Damage Equivalent Loads

The DEL method involves the calculation of a damage equivalent load defined as the constant amplitude load range producing constant amplitude stress ranges that theoretically would result in the same DCR found using Miner's Rule summation.

The calculation and use of a damage equivalent load (DEL) assumes that the incremental fatigue damage of a particular load range from a simulation can be converted to an equivalent incremental load range occurring with a different number of cycles. Each load range is converted to an incremental DEL by assuming a constant slope log-log relationship between load ranges and cycles of loading. The DEL concept may be expressed by the following equation:

$$DEL = \sum \left[\frac{n_i (\Delta R_i)^m}{N_d} \right]^{1/m} \qquad \text{(Eq. 7-21)}$$

where:

i = load range number

n_i = number of cycles of load range ΔRi

ΔR_i = load range for range i

m = assumed slope of log-log relationship

N_d = selected number of cycles to determine DEL

Various slopes are assumed for the log-log relationship. The assumption is that the slope that most closely represents the S-N curve for the components under investigation will result in the most accurate DCR. Typically a loads document provides DEL values based on a range of assumed slopes. It is common to use the DEL calculated with a slope of 4 when investigating direct stresses and a slope of 6 when investigating shear stresses.

The DEL is applied to the supporting structure to determine the stresses in the critical components. The stresses are considered constant amplitude stress ranges and are compared to allowable S-N curve stress ranges for the number of cycles used to determine the DEL. The DCR becomes the ratio of the calculated stress range to the allowable stress range, or alternately with the same result, the ratio of the number of cycles used to determine the DEL to the allowable number of cycles for the calculated stress range.

Although the DEL method is considered a more approximate method, it has the advantage of allowing a simple analysis of a supporting structure for determining conformance to local building code requirements.

The DEL used for an analysis should be based on a number of cycles that will not fall within the fatigue threshold zone of the S-N curves for the components under investigation. If this occurs, an erroneous conclusion may be made that a component has adequate fatigue life if the calculated constant amplitude stress range is below the fatigue threshold stress range. If a lower number of cycles were used to determine a higher magnitude DEL, the computed constant amplitude stress range may fall above the S-N curve for the component indicating inadequate fatigue strength.

For steel structures, it is recommended that DEL's reported in a loads document that are based on more than 2 million cycles be converted to a higher magnitude DEL at 2 million cycles. This will avoid the issue of the calculated constant amplitude stress ranges falling into the fatigue threshold zone for structural components and details typically used in WTGS (refer to Figure 7-1). Another advantage is that the resulting calculated constant amplitude stress ranges determined using the DEL at 2 million cycles may be directly compared to the EN S-N detail categories.

Since a uniform slope is assumed, the DEL from a loads document may be converted to an equivalent DEL at 2 million cycles using the following equation:

$$DEL_2 = (DEL)\left[\frac{N}{2x10^6}\right]^{1/m} \qquad \text{(Eq. 7-22)}$$

where:

DEL_2 = the damage equivalent load based on 2 million cycles

DEL = the damage equivalent load reported in a loads document

N = the number of cycles used for the determination of the DEL

m = the slope used for the determination of the DEL

7.4 Special Analysis by Finite Element Analysis (FEA) Methods

When required by the *Certification Agency Rules* or the *Engineer*, the special analysis may be performed in accordance with the following sections.

7.4.1 Top Flange Eccentricity Analysis

When connected to the tower top flange, turbine yaw bearings are known to load the top flange in a geometrically eccentric manner. These load eccentricities are not necessarily accounted for in a typical analysis based on "mirrored flange" contact. The stresses induced by eccentric yaw bearing loads may be investigated using the methods described in [Freese, 2000].

7.4.2 Hotspot Analysis at Shell Penetrations

Hotspots, i.e., locations of stress concentration, occur at tower shell penetrations, abrupt change in cross section and other geometric discontinuities, other stress risers, etc. Hotspot stresses may be determined in accordance with [IIW, 2003].

7.4.3 Buckling Analysis

The effect of shell penetrations on tower buckling capacity may be determined in accordance with *Certification Agency Rules*. In the absence of such rules, the buckling analysis procedures in EN 1993-1-6 may be used. Alternatively, any rational analysis procedure may be used, but due consideration should be given to nonlinear effects, in the absence of which buckling capacity may be overestimated. A procedure that considers geometric and material nonlinearity with imperfections in the shell's initial shape would be acceptable.

7.4.4 Section Splice Connections

7.4.4.1 Bolted Splice Flanges

Bolted splice flanges may be designed in accordance with *Certification Agency Rules*, where applicable. At this time, this *Guideline* recognizes no US design standard that is sufficiently specialized to serve as a design basis for bolted splice flanges. For this reason, FEA is recommended as a possible design option with recognition that flange manufacturing tolerances, gaps, and imperfect contact reduce the real strength of the joint. Alternatively, the following European documents may serve as a design basis for the strength and fatigue design of bolted splice flanges: [Petersen, 1998], [Schmidt, 1997], and [Seidel, 2001].

7.4.4.2 Alternative Connections

Alternative tower section splice connections such as bolted shear connections or field welded joints should be designed in accordance with the standards applicable to similar connection details in the tower.

7.5 Tower Internal Components

In addition to the tower primary structure (e.g., tower shell, splice and base flanges, and shell penetration reinforcement), the tower internal components often include miscellaneous structural details such as service platform framing, connection and support brackets, ladders, equipment and cable supports, service lift carrier beams, stairs, handrails, guardrails, etc. Some tower internal components may be governed by the *local building code*, and the *Authority Having Jurisdiction* may require structural documentation for these items.

In practice, it is common for tower internal components to fall under separate design scope from the primary tower structure. The connection to the tower wall is the interface between the primary tower structure and the miscellaneous tower internal components. In these instances, the design of connections to the tower should be considered part of the tower internal component design, and the tower internals *Engineer* should verify that the specified connection is compatible with the fatigue detail category of the tower wall.

7.5.1.1 Connections to the Tower Wall

In addition to considering the required strength of the connections to the tower wall, the connection *Engineer* should determine that the connection detail is compatible with the fatigue detail category of the tower wall. Where subject to fatigue loading, the fatigue resistance of the attachment itself should also be evaluated.

7.5.1.2 Platforms

Local building code loading requirements should apply. For determining required live load, service platforms are generally not considered to be part of an exit pathway. The displacement criteria for platform members should be at the discretion of the *Engineer*.

The design of steel platform framing and decking should meet AISC design requirements. Aluminium components should meet [AA, 2000] design requirements. The contact between dissimilar metals should be separated to prevent galvanic-series corrosion.

7.5.1.3 Ladders

Local building code and/or Occupational Safety and Health Administration (OSHA) requirements should apply. All ladder components including support brackets and connections in the load path should be designed to meet code loads and the required fall arrest system forces. Where the ladder system also supports cables or equipment, those loads should be considered in the design.

7.5.1.4 Stairs, Handrails, and Guardrails

Local building code and/ or Occupational Safety and Health Administration (OSHA) requirements should apply.

7.5.1.5 Other Support Framing

Beams and other support framing should be designed to meet AISC requirements if steel or AA requirements if aluminium. The connections to the tower wall should meet applicable strength requirements and should be compatible with the fatigue detail category of the tower wall.

7.5.1.6 Tuned Mass Dampers

Where Tuned Mass Dampers (TMD) is used for vibration mitigation, it may be designed in accordance with the methods outlined in [Faber, 2008] or other rational methods at the discretion of the *Engineer*. Where displacement due to extreme wind loads or design earthquake forces exceed the TMD's rated displacements for effective damping, the TMD may be assumed to be ineffective and only the TMD mass need be considered in the design.

7.5.1.7 Internal Chambers

Internal chambers created by welding steel plate and bulkheads to the tower wall should be verified to be consistent with the tower wall's fatigue detail category. Examples of internal chambers include sand chambers, oil or coolant reservoirs, spill containment floors, internal bulkhead or divider walls, etc.

7.6 Inspection and Testing Requirements

Where compliance with *Certification Agency Rules* is required, the testing and inspection requirements of the *Certification Agency* should apply.

Where compliance with *local building code* is required, the testing and inspection requirements of the *local building code* should apply. Where the *local building code* provides no guidance, IBC Chapter 17 should serve as the basis for the minimum inspection and testing requirements.

7.7 Coordination with Local Building Code

Since the international standards such as IEC 61400-1 and the *Certification Agency Rules* represent a more detailed and specialized design basis for wind turbine support structures, it is recognized that compliance with the regulatory requirements of the *local building code* may not necessarily require the extent of technical detail and rigor contained in the specialized standards. For this reason, the following design assessments are recommended to provide a baseline design assessment, especially for *AHJs* whose primary goal is the structural design review of the support structure to determine *local building code* compliance. The following design assessments are recommended:

1. Frequency separation: While not a typical concern for building-type structures, avoiding resonance is a primary design concern for *WTGS* support structures. See Section 5.4.6.

2. Wind design strength: Design strength against extreme wind load combinations may be calculated in accordance with Section 7.2. The *local building code* design wind may be reconciled with IEC 61400-1 site class ratings as indicated in Section 5.4.7 and 5.4.8.

3. Earthquake design strength: Design strength against seismic load combinations may be calculated in accordance with Section 7.2. The *local building code* design earthquake requirements may be reconciled with IEC 61400-1 earthquake requirements as indicated in Section 5.4.4.

4. Fatigue strength: Fatigue strength may be evaluated in accordance with Section 7.3.

5. Inspection and testing: Inspections and testing requirements may be evaluated in accordance with Section 7.6, **Error! Reference source not found.** and **Error! Reference source not found.**.

8 Foundations

At this time, *local building codes* are not sufficiently specialized for *WTGS* foundation design and as such should be supplemented by *Certification Agency Rules* and other international codes deemed better suited for a particular design aspect. Nevertheless, in the current regulatory process of structural permitting, a *WTGS* project may be subject to the design requirements of the *local building code*. For the purpose of complete foundation design, full compliance with *Certification Agency Rules* is recommended as a design basis, followed by design validation against *local building code* requirements. The required extent of the design assessment with respect to satisfying the *local building code* can only be determined by the *Authority Having Jurisdiction*. Any lesser standard or provision from *Certification Agency Rules* should not be used to undercut or violate the *local building code* requirements, unless the *Engineer* has been granted permission to do so by the *Authority Having Jurisdiction* or an agency with the appropriate authority.

Similarly, any lesser standard from the *local building code* should not undercut or violate the *Certification Agency Rules*. If the situation is unavoidable, the *Engineer* should report the conflicting condition to the owner or client. In the event of conflicting standards, the *local building code* should prevail; however, it is the owner's or client's responsibility to determine the effect of *local building code* compliance on the conditions of certification.

8.1 Materials

The following lists represent materials currently in common use in the wind industry. This list is not exhaustive, nor should it be construed to prohibit unlisted but otherwise suitable materials.

Reforcing:
- ASTM A 615

Cement:
- ASTM C 150

Aggregates:
- ASTM C 33

Fly Ash and Other Pozzolans:
- ASTM C 618

Air Entraining Admixture:
- ASTM C 260

Chemical Admixtures:
- ASTM C 494

Embedment Plate:
- ASTM A 36
- ASTM A 572
- ASTM A 588

Anchor Bolts:
- ASTM A 615
- ASTM A 722
- DIN 931
- DS-EN ISO 898-1

8.2 Limit States

Foundations should be designed or evaluated for ultimate limit states, serviceability states and fatigue limit states. Loading and factored load combinations applicable to various limit states for foundation design are those covered in Table 2, 3 and 4 of Section 5.4.3, and seismic load combinations of Section 5.4.4. . The IEC 61400-1 Standard outlines specific operating and loading conditions for investigating fatigue limit states. The simulators generate turbulent winds that allow the determination of the load ranges and number of cycles for moments and shears at tower base that can be expected over the life of the WTGS. The results are specific to a given WTGS. The load factors and factored load combinations for foundation design are typically specified in the foundation load document supplied by turbine manufacturers. A number of more specific recommended practices for meeting the requirements of foundation limit states are discussed in Sections 8.3 and 8.4.

8.2.1 Ultimate Limit States

Foundation structural elements should be proportioned and designed to have adequate strength to resist the most critical factored load combinations to ensure the structural safety of the foundation. Ultimate limit states of structural elements include ultimate strength of concrete, reinforcing steel, anchor bolts, prestressing elements, grouts and embedment rings. Ultimate limit states may also include stability against overturning, stability against sliding, soil bearing capacity, ultimate axial capacity of piles, drilled shafts and rock anchors, and lateral capacity of piles and drilled shafts. However, the non-structural elements are more typically designed on the basis of allowable capacity under non-factored loads.

8.2.2 Serviceability Limit States

All foundations should be analyzed to verify their serviceability under operation loads is met. Serviceability limit states may include foundation settlement, tilt, ground gapping, foundation stiffness, crack width, soil cracking and foundation movements.

8.2.3 Fatigue Limit States

Fatigue analysis should be performed to verify that concrete, reinforcing steel, prestressing steel, anchor bolts have adequate fatigue strength to resist the cyclic fatigue loads prescribed by wind turbine manufacturer. More specific recommended fatigue evaluation guidelines are discussed in Section 8.4.2.5.1.

8.3 Anchorages

Tower anchorages have historically consisted of two types: embedded and bolted. Embedded tower anchorages comprise a short section of tower that is cast into the reinforced concrete foundation and then bolted to the remainder of the tower via a conventional tower flange-to-flange connection. Bolted tower anchorages comprise bolts attached to a flange at the base of the tower that are terminated in the mass of the reinforced concrete foundation using a steel ring plate, washers, and nuts. The bolts are commonly designed with post-tensioning and the flange is typically of a tee configuration that is welded to the tower shell. The tee rests atop a bed of grout which is used not only to level the tower during erection, but also to accommodate the very high stresses imparted by the tower base flange as a means of transition to the lower strength concrete in the foundation below. Spreader plates have been used to transition stresses from the tower base flange to the grout. Other anchorage configurations may be possible, but the foregoing has dominated *HAWT* tower anchorages for the past 25 years.

8.3.1 Embedded Anchorages

Embedded anchorages, with respect to the short tower portion, are subject to all the considerations and requirements of tower design. The method of embedded anchorage load transfer to the reinforced concrete is subject to conventional reinforced concrete design practice with due consideration for fatigue. The absence of preload mechanism, with respect to cyclic concrete stresses of changing sign, should be recognized and is often addressed in practice through means of provision of tensile load path in reinforcing only. This often leads to amounts of reinforcing that would exceed amounts anticipated based on ultimate stresses alone.

8.3.2 Bolted Anchorages

Bolted anchorage design includes the following elements:

- base flange
- grout beneath base flange
- concrete beneath the grout
- bolts
- washers
- nuts
- embedded ring plate

The listing is provided to highlight the potential interdependence of tower components with the remainder of the anchorage design, chiefly in consideration of prying forces and bolt / flange hole eccentricities which can introduce potentially damaging excess stresses into the various elements. Respecting this interplay turbine tower and base flange requirements such as life-cycle anchor bolt post-tension force and minimum bolt diameter should be recognized and included in specifications and design.

Apart from the above the tower and base flange elements are subject to all the considerations and requirements of tower design. The remaining elements are subject to conventional steel and reinforced concrete design practice with due consideration for fatigue.

In recognition of the high fraction of overall joint stiffness that can be attributed to the reinforced concrete, as well as the significant damage that can occur in concrete subject to cyclic loading with a high incidence of unloading bolted anchorages are almost exclusively designed as post-tensioned.

8.3.2.1 Grout

Grout under the tower base flange should be designed to resist the applied loads with due consideration for fatigue including initial loads (post-tension force) in the anchor bolts. The designer should specify the required permanent strength as well as the strength required during construction (e.g. tower / turbine erection and anchor bolt post-tensioning). Grout should be designed or detailed in consideration of the interface with the tower base flange and service climatic conditions such as precipitation, freeze/thaw cycling, and use of de-icing chemicals. Reference is given to ACI 318-08, 351.1R-99 and 351.2R-94.

8.3.2.2 Anchor Bolts

Anchor bolts should be designed according to applicable standards for steel construction [AISC, 2005] with due consideration for fatigue loading, corrosion protection, the stiffness of the bolted tower / reinforced concrete joint, and the load share of each of the elements (concrete and bolts). Bolt toughness should be considered for inclusion in material specification although it should be noted that the general state of practice is for there to be no toughness requirement. Because, in the instance of post-tensioned bolted anchorages, joint durability depends on maintenance of anchor bolt post-tension force a bolt post-tension force monitoring and maintenance program should be specified.

In practical terms the anchor bolt post-tension force specified is often set equal to 15 to 20% above the worst computed bolt tributary load from the tower for the extreme unfactored tower base overturning moment. Normally without need for further checking of the joint element load share, etc. this has been shown to be a reliable method of achieving a post-tension level that ensures compression in the concrete for the full range of operational loads while limiting stress ranges in the bolts to ensure their fatigue resistance. Certain foundation geometries and levels of post-tensioning would not necessarily ensure the limits to concrete and bolt stress as discussed above, however, and these therefore should be evaluated in keeping with the above general guidance.

It should be cautioned that loss of bolt preload from creep can be excessive if galvanized coating used on the faying surfaces of heavy plates exceeds 5 mils. High bolt preload will cause excessive creep and loss of bolt pretension.

8.3.3 Anchorage load transfer

Anchorage shear and moment load transfer to the remainder of the foundation should be ensured for ultimate and fatigue loading.

The regions of pre-stressed and non-prestressed concrete and reinforcing should be identified and designed accordingly.

Little research is available on the performance of anchorages of the type and size found in modern utility-grade HAWT foundation construction.

8.3.3.1 Shear / Pullout

With respect to design for shear, often referred to as pullout, of the anchorage a variety of methods are currently in use. One method used by designers in the industry is found in the provisions of Chapter 6 of [PCI, 1999]. Another method employed includes [ACI, 2008] Chapter 11 provisions for columns and slabs that evaluate different numbers of angular sectors of the anchorage for vertical loading to seek the worst pullout condition. Yet another method is to consider anchorage vertical reinforcing as that of a round column and make evaluations per [ACI, 2008] Chapter 10 provisions in overturning. Due the differing strain levels at which concrete and mild reinforcing will reach their respective ultimate capacities in the subject manner of shear / pullout, if reinforcing steel is added to resist pullout of the anchorage it should be designed to resist the entire pullout force, as discussed in commentary section RD4.2.1 of [ACI, 2008], Appendix D and Chapter VIII of [PTI, 2006].

All of the above include efforts to ensure the development of added reinforcing into both the mobilized anchorage and the remaining foundation.

Design for fatigue in shear / pullout of the post-tensioned anchorage has been absent or inconsistent throughout the industry in the U.S. AISC Design Guide 1 and Transportation Research Board NCHRP Report 412 provide useful guidelines for fatigue evaluation of anchor bolts.

8.3.3.2 Moment

Adequate ability to transfer the applied moment from the tower anchorage to the foundation should be ensured for ultimate and fatigue loading according to recognized moment transfer methods such as those described for slab-column connection by the latest edition of [ACI, 2008] with due recognition for the appropriateness of methods selected respecting the size of the footing elements and the availability of test data to underpin design methods.

When design moment from the tower anchorage exceeds bolt pretension, anchor bolts are subject to combined shear and tension. Anchor bolts and concrete embedment may be designed using ACI318 Appendix D. Additionally, ASIC Petrochemical Committee report on Anchor bolts provides useful guidelines for design of post-tensioned anchor bolts.

8.4 Considerations Specific to Certain Types of Foundations

8.4.1 Shallow Foundations

Shallow foundations are defined in this context as a foundation system relying on dead weight to resist overturning loads. These foundations have numerous unique concerns as detailed below.

8.4.1.1 Foundation stiffness

Foundation stiffness requirements are of very high importance and in some instances may control the design of the foundation. If the stiffness requirements are not met by the design, the turbine's expected fundamental frequency may be different than anticipated during tower design. Overall foundation stiffness depends on the strength and stiffness parameters of the soil, and their interaction with the structural elements of the foundation. It is common for turbine manufacturers to specify minimum rotational and/or translational stiffness values for wind turbine foundations. If specified, the stiffness of the foundation can be calculated assuming the soil is an elastic half space, or a semi-infinite continuum of soil idealized as an elastic material. The shear modulus of the soil should be determined from measurements taken at the project site from the geotechnical report. This small strain shear modulus should be reduced for the strain calculated or estimated to result from the wind turbine loading. Guidance for performing this calculation can be found in [DNV/RIS0, 2005].

8.4.2 Differential settlement or tilting

Total and differential settlement should be kept to an acceptable level. Settlement should be calculated for the entire foundation influence zone and include immediate settlement, primary and secondary consolidation settlement, as well as seismically induced settlement. In the absence of limits specified by the turbine manufacturer, a maximum inclination of 3mm/m may be assumed.

8.4.3 Bearing Capacity

The foundation support material should be evaluated to determine the ultimate and allowable bearing capacities. The allowable bearing capacity should include an adequate factor of safety per requirements of the applicable building code. In the U.S., bearing capacity is traditionally evaluated with allowable stress design approaches. Per [IBC, 2009], the factor of safety should be at least 3.0 in determining the allowable bearing capacity at service loads and fatigue loads, and 2.26 under unfactored extreme loads. Evaluation of bearing capacity should also include consideration of eccentric loading due to the turbine overturning moment and the resulting reduced foundation contact area.

Evaluation of bearing capacity should take into account all soil layers that are within the influence zone of the foundation as determined by bearing capacity theories.

When calculating bearing capacity, the following considerations should be included:

- Eccentricity of the foundation/ Effective foundation area
- Design Groundwater Level
- Drained conditions
- Undrained conditions
- Bearing capacity factors
- Ultimate limit state approach (ultimate strength or Load and Resistance Factor Design) may be used for evaluation of foundation soil bearing capacity if it is permitted by the applicable building code.

8.4.4 Overturning Resistance

The foundations should be designed to provide adequate resistance to overturning due to loads imposed by the wind turbine and other conditions such as earthquakes. The required resistance level should be consistent with local building code requirements but the factor of safety (Resisting Forces/ Unfactored Overturning Forces) should not be less than 1.5. Generally only the dead weight of the structure, foundation, and backfill materials should be considered in analysis of overturning resistance. If passive or shear resistances are considered they should be justifiable considering the degree that they can be mobilized before overturning could occur.

8.4.4.1 Ground Gap or Zero Pressure

8.4.4.1.1 Permanent Loads

Under unfactored permanent or normal operating loads, contact pressure should be compressive under the entire foundation; i.e., no ground gap or zero pressures should occur. This ensures that the foundation stiffness remains adequate during normal operation loads and contributes to reducing the cyclic degradation of the foundation bearing materials. Permanent loads are defined in [GL, 2010] as DLCs 1.1 (power production under normal turbulence model) and 6.4 (parked under normal turbulence model) with a probability of exceedance of 10-2 (equivalent to 1750 hours in 20 years).

8.4.4.1.2 Extreme Loads

Under unfactored extreme loads, the ground gap shall extend no further than the center of gravity of the foundation.

8.4.5 Sliding Resistance

The foundations should be designed to provide adequate resistance to sliding due to loads imposed by the wind turbine and/or conditions such as earthquakes. The required resistance level should be consistent with applicable building code requirements but the factor of safety (Resisting Forces/Unfactored Driving Forces) should not be less than 1.5. Only the dead weight of the structure, foundation, and backfill soils above the foundation should be considered in analysis of sliding resistance.

8.4.6 Load Factoring

The foundation should be designed to resist the internal forces and moments resulting when the factored loads are applied to the foundation, as stipulated in section 15.2.1 of [ACI, 2008]. Note that the resulting forces and moments may be significantly different than those resulting from applying the load factor to the forces and moments resulting from the unfactored loads.

8.4.7 Reinforced Concrete Design

Reinforced concrete should be designed per ACI 318-08 (or latest) building code for strength, serviceability and durability. Special attention should be given to prevent pedestal pullout, provide adequate moment and shear transfer at pedestal/slab junction, keep bearing stress in the concrete and grout at the tower flange/ foundation interface within code limit, analysis of bursting forces in the post-tensioning anchorage zone and determination of the required reinforcement. Additionally, proper effective slab width considering stress concentration should be used in calculation of flexural moment and shear demands of the foundation mat.

Note that IEC 61400-1 contains loads and load cases related to conditions not contemplated by [ASCE, 2005]. Additionally, strength reduction factors of [ACI, 2008] differ from the partial safety factors of the design codes recommended by [IEC, 2005]. Future research on the inherent reliability assumptions of [IEC, 2005] and [IBC, 2009]/[ACI, 2008] is required to reconcile the differences between the various codes. Until this research is available, it is left to the designer to ensure that the intended reliability of each of the different codes is met.

In current practice, the foundation loads are calculated according to [IEC, 2005] by the turbine manufacturers and provided for the foundation designer's use. Most foundation designers in the industry design the foundations to meet the requirements of [ACI, 2008] as a minimum. [IEC, 2005] requires that, when other national standards are used to calculate the capacity of members, the designer should ensure that the design results in a level of safety consistent with the standards intended by [IEC, 2005], (presumably the Eurocode). Some turbine manufacturers provide guidance on the topic and require different load factors to compensate for a perceived difference in reliability between [ACI, 2008] and what [IEC, 2005] requires. Similar measures may be required when a project requires certification by an entity such as [DNV, 2005] or [GL, 2010] which requires the use of different national standards.

8.4.8 Fatigue Analysis

Fatigue adequacy verification for concrete structures should be performed for both the concrete and for the reinforcement in separate analyses. In the absence of applicable U.S. building codes/standards, fatigue evaluation may be performed in accordance with one of the following referenced standards/codes: [DNV, 2007], Eurocode 2 and 3, CEB-FIP Model Code 1990 or [GL, 2010], unless turbine manufacturer has its specific recommendations. For fatigue analysis, the partial load factor on loads should be taken as 1.0 but additional factors should be applied per the standard used. It is recommended that the foundation fatigue evaluation comply with the fatigue criteria as defined in one of the above standards/codes. Partial safety factors for fatigue loads, materials, safety class and fatigue damage should be no less than those defined in the standards/codes, and in no instance should overall safety level for fatigue be less then as prescribed per the turbine certification standard [IEC,2005].

8.4.9 Deep Foundations

Deep foundations include drilled piles, drilled shafts and pier foundations that are post-tensioned or regularly reinforced, pile groups supporting concrete caps, rock and soil anchors and other proprietary foundation systems such the tensionless pier (mono-pier) foundation. Piles include driven piles, drilled shafts, bored piles, auger-cast piles, and micropiles. Other types of deep foundations not specifically mentioned herein may be used, provided that they can be substantiated by acceptable test data, calculations and other information relating to the structural properties and load capacity of such elements. Where building permit is required, the use of special type of deep foundations is also subject to the approval of the building official.

8.4.9.1 Safety Factors

Safety factors shall be applied to foundation types to determine allowable capacity based on design assumptions used and confirmation testing conducted during installation (for driven piles). Guidance for global and partial safety factors for deep foundations is provided in chapter 18 of [IBC, 2009]. The factors of safety specified in IBC are appropriate for service loads and not for extreme loads. Under unfactored extreme loads, a minimum of factor of safety of 1.5 is recommended. Additional guidance is provided in other references. Partial resistance factors for a limit states design approach can be found in Transportation Research Board NCHRP Report 507.

8.4.9.2 Mono-Pier Foundations

The mono-pier foundation (such as the tensionless pier) resists the applied horizontal loads and overturning moment mainly by horizontal passive resistance and vertical skin friction of the earth materials that surrounds the pier and to a much lesser extent by bearing on the base of the pier. The passive and shear resistance relationships of the earth materials shall be based on rational methods.

8.4.9.3 Post-tensioned, rock and soil anchor foundations

Post-tensioned, rock and soil anchor foundations consist of an upper reinforced concrete mat that have anchors installed by drilling a shaft and filling the shaft with a high strength anchor bolt and grout. The anchors are post-tensioned to develop an internal tension force in each anchor that is locked off by a nut bearing on a base plate atop of the concrete mat. Overturning moment loads are transferred through the concrete mat to the subgrade and anchors by soil structure interaction that requires evaluation of the stiffness and strength of the subgrade and bond strength of the grout/earth interface.

8.4.9.4 Foundation Stiffness

Foundation stiffness for deep foundations should be determined using soil structure interaction analysis or other suitable procedures.

Stiffness of single pile, pier and rock anchor can be determined on the basis of testing, or computer aided soil structure interaction analysis. Where piles or piers anchors are connected to a concrete cap, it should be demonstrated that the piles or piers anchors alone have adequate stiffness required for the foundation. The stiffness reduction due to group effect of piles and piers should be included in the determination of foundation stiffness.

The mono-pier foundation behaves as a very large diameter, short, rigid pile. The stiffness of the foundation which relies on both horizontal and vertical restraint of the earth materials surrounding it should be evaluated using finite element method or other appropriate methods.

Soil stiffness is strain-dependent and effects of strains on soil dynamic stiffness properties should be considered. Where geotechnical investigation indicates soil at site is susceptible to cyclic stress degradation, the reduction in soil stiffness properties should also be included in the foundation stiffness determination.

8.4.9.5 Pile Fatigue

It should be demonstrated through fatigue analysis that the piles are capable (both geotechnical and structural fatigue resistance) of withstanding the number of cycles expected during normal operation of the turbine.

8.4.9.6 Anchor Fatigue

It should be demonstrated through fatigue analysis that the foundation anchors, including rock and soil anchors, are capable of withstanding the number of cycles expected during normal operation of the turbine considering the benefit of post-tensioning to reduce the cyclic stress fluctuation in the anchors. The reduction in stress fluctuation is dependent on the relative stiffness between the anchor system and the subgrade. Acceptable fatigue checks may be performed per this Guideline.

8.4.9.7 Overturning Resistance

All deep foundations should be demonstrated by analysis that the foundation has adequate overall resistance to overturning moments (including the effects of lateral shear and torque).

For pile and pier foundations, the overturning resistance should be determined on the basis of allowable capacity of pile/piers including the group effects (refer to section below). The bearing resistance under concrete cap should not be included in the calculation of the foundation overturning resistance. The overturning moments induced by unfactored extreme wind loads should not exceed the allowable overturning resistance. Overturning resistance of pile and pier foundations connected to a structural mat should be evaluated considering the rotational restraint that the pile/pier group provides in the allowable axial tension and compression capacity of the piles or piers.

For the mono-pier foundation, the overturning resistance should be taken as a combination of passive earth resistance (above and below an equilibrium point of rotation), vertical side shear along the length of the foundation, shear resistance at the base and axial resistance at the base. The ultimate passive earth resistance and shear friction is derived from the principles soil and rock mechanics. A global safety factor against the unfactored, extreme wind load of at least 2.0 should be provided.

8.4.9.8 Axial Pile and Pier Capacity

Pile and pier foundations should be designed to provide adequate capacity for axial loads imposed by the turbine. The design should demonstrate adequate skin friction to resist axial loads. End bearing resistance may be included in evaluating axial capacity depending on the pile or pier installation method or at the discretion of the Engineer.

Where applicable, the effects of *settlement and negative skin friction (*downdrag*)* should be accounted for in axial capacity calculation.

The best method for determining actual installed pile capacity is by static load test. Verification of pile capacity, when required by [IBC, 2009] or by project specification, may be conducted during installation based on dynamic measurements and/or pile wave equation analyses with prior approval by the Engineer. Dynamic formulae, (such as the EN formula), are not considered an accurate predictor of pile capacity.

8.4.9.9 Lateral Capacity

The lateral load carrying capacity of deep foundations should be determined using appropriate methods. Where beam on nonlinear elastic foundation method (e.g. p-y) is used, it should be applied appropriately with material properties representative of the foundation support materials. Additionally, verification of lateral load carrying capacity by load tests may be required for driven piles per [IBC, 2009].

8.4.9.10 Structural Design of Deep Foundations

Concrete cap (mat) and other concrete elements (piles, piers) should be designed to comply with the strength, serviceability and durability provisions of [ACI, 2008]. Guidance for load factors is found in Section 8.6.1.9 of these guidelines. Alternatively, the deep foundation elements (steel piles, micropiles, piers) may be designed using the allowable stresses not exceeding those specified in Table 1810.3.2.6 of [IBC, 2009].

Where post-tensioning is utilized, foundations should be designed using the recommendations and requirements for prestressed concrete in the [ACI, 2008] and [ACI, 1993] as applicable. The analysis of the concrete and anchorages that comprise the foundation should consider: determination of the required post- tensioning forces that confirms that the foundation remains in compression; check of tension in the anchorages; analyses of the bearing stress in the concrete and grout at the tower flange/ foundation interface; analysis of bursting forces in the post-tensioning anchorage zone and determination of required reinforcement to resist said forces; and analysis of shear in the overall concrete section.

8.4.9.11 Group Capacity

Pile groups shall include analyses of group interaction and modifications to group axial and lateral capacity. Contribution to lateral capacity from an embedded pile cap may be considered provided that it can be demonstrated that the pile cap can be sufficiently embedded to provide lateral resistance under all applicable loading conditions. For grouped deep foundation elements, the allowable working uplift loads should be calculated to meet the provision of Section 1810.3.3.1.6 of [IBC, 2009].

8.4.9.12 Corrosion and Soil Erosion

Deep foundations shall be evaluated for corrosion of structural elements in contact with subsurface materials. Unless fabricated of corrosion resistant materials, corrosion evaluation for steel piling and pile cap connections should be performed in accordance with section 2203 of [IBC, 2009]. Consideration should also be given for corrosion of concrete foundation elements in accordance with ACI guidelines.

Where deep foundation is subject to erosion, depths of soil erosion or scouring should be considered based on appropriate hydraulic study.

8.4.10 Rock Anchored Foundations

8.4.10.1 Overturning Resistance

Overturning resistance for the post-tensioned, anchor foundation should be checked against the design overturning moment similar to a shallow foundation except that a portion of the restraining force due to the anchor post tension may be included in the resisting loads. The number of anchors mobilized for overturning resistance should be determined based on rational methods. A minimum factor of safety of 2 should be provided for overturning resistance at unfactored loads.

8.4.10.2 Bearing Capacity

The foundation support material should be evaluated to determine the ultimate and allowable bearing capacities in a similar fashion to a shallow foundation.

8.4.10.3 Axial Anchor Capacity

The geotechnical capacity of the anchor should be checked against pull-out with respect to the design tension load. The capacity should be evaluated based on representative values of the bond stress between the grout and the surrounding rock. Bond stress provides the primary mechanism for resisting pull out and is dependent on the rock type and characteristics (strength, rock mass modulus, weathering, discontinuities, etc.), and the method of grouting. The design bond stress should provide a minimum factor of safety of 2 for the rock/grout interface

The length of anchor within the rock that will resist the tension load is the bond length. After the bond length is determined, a calculation should be performed to check that there is enough soil/ rock mass above the bond length to resist the design loads assuming a global failure of the rock mass. The global rock mass failure zone may be assumed to be an inverted cone with an apex angle of 60 degrees propagating from the middle of the bond zone to ground surface, unless a different angle is justified through a rational analysis. The anchor design should also include an adequate unbonded/stressing length to allow for re-stressing of the anchors.

The structural capacity of the anchor is limited by the allowable tension load on the anchor taken as 70% of ultimate strength of the anchor rod.

Guidance for design of rock or soil anchors can be found in the [PTI, 2005], and other documents such as [FHWA, 1999].

8.4.10.4 Anchor Load Testing

All anchors should be tested in accordance with [PTI, 2005]. All anchors should be proof-tested to 133% of the design post-tensioned load before lock-off. At least one anchor per foundation will be performance tested per PTI procedures with load and reload cycles.

The active length of the anchor is dependent on the distribution of transfer of bond stress and skin friction along the length of the anchor. At minimum, the equivalent elastic length of the anchor is the unbonded length of the anchor, but at maximum it should not exceed the unbonded length plus one half of the mobilized bonded length.

The visco-elastic creep at the anchor grout/ bond may be a concern depending on the type of earth materials encountered.

8.4.10.5 Design Post-Tension Load to Anchors

The long-term, effective, post-tension load should be determined so that the concrete cap remains in contact with the subgrade during operational loads. The design lock off post-tension load should account for tension losses from visco-elastic creep.

8.4.10.6 Post Tensioned Anchor Foundation Mat Structural Design

The structural design of the reinforced concrete cap is similar to that of a shallow foundation except that the anchors provide points of restraint.

9 Fabrication and Installation

9.1 Scope

Fabrication and installation generally do not fall under the EOR's scope of work. The information contained in this section is with regards to the verification activities for the tower and foundation support structures during fabrication, pre-construction and during installation/construction. Inspections during commissioning and operational phases of the wind farm are discussed in Section 10. The information provided in this section should not be construed to subsume these items (i.e., the means and methods of the Fabricator or Field Contractor) under the responsibility of the design *Engineer*. Engineering design responsibility and the associated design liability do not include the means and methods implemented by the tower Fabricator. Similarly, neither does engineering responsibility extend to the means and methods implemented by the erection or Field Contractor in installing the tower or building the foundation.

Nevertheless, in situations of real practice and as a basic matter of project cooperation, the Engineer is often called upon to provide engineering advice on related fabrication and installation issues, when requested by the various project players. The Engineer should preferably proceed in these cases only with sufficient understanding of the boundaries of individual liability and knowledge. In light of these conditions, the purpose of this section is to provide practical information regarding the common intersections of the otherwise separate scopes of work of tower/foundation engineering, fabrication, and installation/construction operations. In particular, items and issues that may affect the design life or design safety factor are specifically discussed.

9.2 Tower Fabrication and Installation

9.2.1 Fabrication Tolerances

It is recommended that the structural design drawings incorporate tolerances requirement explicitly or by reference. Structural design drawings for local building code compliance may not show fabrication tolerances, however, tolerance information shall be required on shop, assembly, or fabrication drawings. Where fabrication tolerances are not shown on the available drawings, the fabricator should coordinate with the Engineer and turbine and/or tower manufacturer to determine the required fabrication tolerances. In the case of conflicting specified tolerances, the fabricator should contact the Engineer and turbine manufacturer. Alternatively, the more stringent tolerance may be used where there is lack of agreement or clarity.

9.2.1.1 Quality Assurance/Quality Control (QA/QC)

This section encompasses the general requirements of tower fabrication QA/QC including but not limited to the following items:

Inspection and Testing Requirements

- Review of material test Certified Mill Test Report (CMTR) or material mill certificates
- Visual inspection of raw or conditioned steel plates
- Review of supplementary tests results including Charpy V-Notch (CVN) tests
- Review of welding documentation such as weld procedure specifications (WPS) and procedure qualification reports records (PQR)
- Visual inspection Testing (VT) of welds
- Inspection of weld preparation
- Nondestructive testing (NDT) of welds by Magnetic- Particle Test (MT), Ultrasonic Testing (UT), or Radiographic Testing (RT) techniques
- NDT for lamellar tearing of plate material at highly restrained welded joints

Acceptance/Rejection Criteria

Repair Procedures

Reporting Requirements

9.2.1.2 Governing Inspection Criteria

Conflicts regarding QA/QC have been known in the wind industry to lead to legal disputes between the Fabricator and other project parties. Such disputes typically stem from the lack of agreement on the governing QA/QC standard. For this reason, it is imperative that project parties undertake a coordinated effort to establish mutually agreed upon governing QA/QC standards prior to tower fabrication.

The difficulty in establishing an agreed QA/QC standard may often stem from the mix of standards involved in the WTGS project, which may involve one or more of the following: Local Building Code, Certification Agency Rules, turbine manufacturer's proprietary tower specifications, Engineer's in-house specifications, and Fabricator's internal standards. Inherent to these standards are the reference to either US standards or European international standards. Adding to the complexity of the situation, the basis in US standards may be substantially different: for example, North American steel tower Fabricators may implement either American Welding Society (AWS) structural welding standards or American Society of Mechanical Engineers (ASME) pressure vessel welding standards. While it is beyond the scope of this Guideline to reconcile the variety of available QA/QC standards, the following sections offer some recommendations that may prove useful.

9.2.1.2.1 Recommendations for Regulatory QA/QC Compliance

Where regulatory compliance with Local Building Code is required, then those test and inspection requirements should apply. Insofar as the IBC serves as the Code basis throughout most of the U.S., compliance with IBC Chapter 17 Special Inspection requirements is recommended for baseline QA/QC criteria. Note that the IBC provisions use AWS/AISC and ACI reference standards for steel and concrete construction, respectively.

9.2.1.2.2 Recommendations for Supplementary QA/QC Compliance

With the exception of required, mandatory regulatory standards, all other QA/QC standards (such as those from Certification Agency Rules, turbine manufacturer specifications, etc.) may be considered to have supplementary status as either commercial or contractual requirements. It is imperative that some coordination effort be undertaken among project parties to establish the extent to which all other supplementary QA/QC requirements become part of the project requirements.

9.2.1.2.3 Recommendations for Alternative QA/QC Compliance

The term "alternative QA/QC" as used herein refers to otherwise supplementary requirements that are proposed in lieu of required standards, of which a common example routinely encountered in industry practice is the use of International standards in lieu of US standards. In such cases, it is recommended that the Engineer provide review and approval in accordance with the "alternative means" or "rational methods" provisions found in most standards. Ultimately, use of alternative standards would require the acceptance of the Authority Having Jurisdiction. While there is no guarantee of agreement, it is assumed that in most cases, the Authority Having Jurisdiction will defer to the judgment of the Engineer supported by adequate documentation to justify a rational substitution of standards. In particular, the Authority Having Jurisdiction should be advised that it is the position of these Guidelines Recommended Practices that the European (i.e., Eurocode) QA/QC standards are: proven in the European wind industry; unofficially "proven" in the US wind industry; and serve as standards for projects in many parts of the civilized world. Therefore, in terms of quality and safety, the International standards should not be viewed with suspicion, but rather as a source of competent and proven "as-equal" standards with minor technical differences.

9.2.1.3 Handling

Tower components should be handled with care in a workmanlike manner. Avoiding even minor damage is critically important with manufactured fabricated tube towers that are thin-shell structures known to be sensitive to local buckling and are subject to high-cycle fatigue loading. For example, a small dent that would otherwise be ignored as insignificant in other steel structures may actually be a pre-buckled condition to structural failure. Also, the tower is a fatigue loaded structure that is sensitive to notches and discontinuities. A scratch or gouge to the tower shell that would otherwise seem minor may be a stress riser or early corrosion location that could serve as a fatigue crack initiation point, greatly reducing the tower's fatigue life. All permanent markings on towers and the method of marking should be approved by the Engineer. Permanent tower markings done by embossing (stamp impression) in the steel plate should be made with approved rounded "low-stress" stamps. Existing permanent marks made by unknown or unverifiable methods should be considered as deep defects and repaired by a combination of welding and grinding at the approval of the Engineer. These and other defects, damage, and repairs in WTGS towers are described in [Agbayani, 2009].

Lifting apparatus such as lifting lugs, spreader bars, temporary tower braces, etc., should be configured to provide adequate support to the lifted tower components and also to minimize and distribute concentrated loading to the lifting points (i.e., "pick points"). Care should also be taken to prevent damage to finished surfaces.

9.2.1.4 Storage

Tower sections should be stored in such a way to prevent corrosion or finish damage and to prevent the build-up of moisture, snow, or mud in the tower interior. Care should be taken to protect from corrosion any raw metal surfaces or designated electrically conductive surfaces such as at grounding brackets or flange contact surfaces.

Tower sections should be stored in such a way to prevent excessive concentrated loading. Support points such as flanges should be temporarily braced to prevent excessive or permanent deformation, unless a properly substantiated engineering analysis has determined that no temporary bracing is necessary.

9.2.2 Tower Installation

For the organizational purpose of this Guideline, tower transport is classified as part of tower installation. It is recognized, however, that the tower transport and shipping logistics going from the factory to the project site may fall under other than the Fabricator's or erection Contractor's responsibility.

Prior to tower installation, the following items should be verified:

- Access path to each tower site to prevent physical damage to tower structure during transport and installation
- Visual inspection of tower structure condition and protective coating following arrival on site.
- Inspection of calibration records for bolt tensioning equipment used to install rods in foundation to tower connection, flange ring bolts between tower sections and other bolts tensioned.
- Bolt tensioning procedures, including sequencing and field bolting records.
- Bill of materials supplied with turbine structure shall be fully reviewed and individual components checked off prior to acceptance with the transportation group.
- Anchor bolt pattern configuration shall be confirmed and within acceptable limitations prior to erection of the turbine structure.

9.2.2.1 Transport

Any QA/QC tower inspections required prior to shipping from the factory should be performed prior to transport. The undamaged condition of tower components should be deliberately established and then clearly documented in coordination with the Fabricator. Insofar as transport insurance is a hedge against economic loss due to transport damage, it still should be proven that the damage occurred during transport rather than during handling at the factory.

In general, the discussion in the previous tower "Handling" section should apply. Resultant forces and reactions on the tower sections should be minimized and distributed to prevent excessive concentration. Care should be taken to protect the tower finish during transport. Shipping supports (e.g., shipping "feet" fixtures, shipping braces, straps, etc.) should meet prescribed design shipping forces and accelerations. Legal, safety, and logistical parameters fall under the responsibility of the tower transport Contractor.

Care should be taken to prevent damage to tower internal components during transport. Factory-mounted internal components (e.g., ladders, platforms, cable trays, or separate packages shipped within, etc.) should be secured against loosening, swinging, shifting, or falling off during transport.

Heavy items such as platforms should be securely braced or blocked against excessive movement during transport. Factory-mounted platforms (which typically occur near section splice flanges) should not unintentionally serve as a tower stiffening diaphragm during transport. Dedicated temporary shipping braces should be used to prevent excessive deformation of tower flanges and tower cross sections during transport.

9.2.2.2 Project Site Storage

In general, the discussion in the previous tower "Storage" section should apply. However, in contrast to storage at a factory yard, it is recognized that shorter term storage at a project site may utilize temporary bracing or support assemblies that are less robust than those used in the factory yard. Nevertheless, any short term field assemblies should meet all necessary functional and safety requirements.

9.2.2.3 Erection

The erection Contractor is responsible for the means and methods of the tower erection. Crane requirements, lifting rigging and apparatus, and safety requirements should be determined by the Contractor. Use of spreader bars, engineered lifting lugs, and temporary flange braces are recommended to prevent the misshaping of flanges. All previous discussions regarding tower "Handling" should apply. The following are issues affecting tower design that can arise specifically during the tower erection process:

- Dents, finish/coating damage, impact, etc.
- Lack of bolt hole fit-up at splice flanges.
- Lack of initial heel contact at splice flanges.
- Residual flange gaps exceeding agreed tolerance.
- Broken high-strength bolts at splice flanges.
- Tower misalignment (out-of-plumb).
- Excessive cross-wind vibration (due to vortex shedding).

All of the above issues affect the tower design life or factors of safety and should be subject to engineering review to determine what repair or mitigation is necessary.

Any QA/QC inspections required just prior to or during erection should be performed at this stage. In particular, the inspection and testing of high-strength bolting should be coordinated between the *Code*-required Special Inspector and the Contractor. A common point of conflict is the requirement by US standards of establishing a torque-to-tension relationship using, for example, a Skidmore-Wilhelm bolt tension calibrator. In contrast, European wind industry practice typically uses torque-only relationships to establish bolt tension. It is recommended that the *Engineer* establish the project QA/QC requirements with respect to any use of alternative means of inspection and testing.

9.2.2.4 Tower Cross-Wind Vibration during Erection

Cross-wind vibration, particularly, vortex shedding may be considered a construction phase issue affecting tower erection. The means and methods of tower erection and erection load cases are not typically considered to be part of the primary tower design scope. These provisions should not be construed to make vortex shedding calculations a mandatory part of the tower primary design calculations. Vortex shedding should be considered in accordance with Certification Agency Rules, where applicable.

The critical wind speeds for vortex shedding should be calculated for susceptible tower configurations. These include all incomplete tower section configurations during tower erection, e.g., base section alone erected, two sections erected, three sections erected, etc.

Configurations consisting of a completed tower plus a mounted partially or fully assembled wind turbine may also be considered for critical wind speed. Time period for fatigue loading from vortex shedding may be in accordance with [GL, 2003]. Calculation of fatigue loading due to vortex shedding may be in accordance with [ASME, 2006] or [DIN 4133].

9.3 Foundation Construction

The success of foundation designs are contingent on verification of soil and conditions assumed during the structural and geotechnical design. This is primarily accomplished through a quality assurance/quality control program carried out during the construction phase of the project. To that end, the following items should be verified during construction, as applicable:

- Soil and bedrock conditions beneath the foundation
- Soil and subgrade shear strength
- Geologic conditions
- Concrete and grout compressive strength
- Backfill shear strength
- Backfill unit weight

The following procedures may be used for construction control and verification of consistent material strength of foundation subgrades and backfills.

- ASTM D 6938 Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods

9.3.1 Concrete and Grout

Concrete works should be in general conformance with the following codes and specifications, and all such requirements of the Engineer:

- ACI 318 Building Code Requirements for Structural Concrete
- ACI 301 Specifications for Structural Concrete
- ACI 309R Guide for Consolidation of Concrete
- ACI 201.2R Guide to Durable Concrete
- ACI 305R Hot Weather Concreting
- ACI 306R Cold Weather Concreting
- ACI 207.1R Guide to Mass Concrete
- ASTM C 94 Standard Specification for Ready-Mixed Concrete
- ACI 351.1R-99 Grouting between Foundation and Support of Equipment and Machinery
- ACI 351.3R-04: Foundations for Dynamic Equipment

9.3.2 Concrete Durability Requirements

Concrete mix design should be in accordance with ACI 318 and take into account the following factors:

- Water-cementitious material ratio
- Freezing and thawing exposure

- Sulfate exposure
- Corrosion protection of reinforcement

9.3.3 Anchor Bolts

Anchor bolt material selection should consider toughness requirements as may be specified by the WTGS Supplier and/or in consideration of cold temperatures at the project site.

9.3.4 Reinforcement

Concrete reinforcement should be fabricated and installed in accordance with ACI 318.

9.3.5 Concrete Placement

Concrete should be placed in accordance with ACI 318 and as recommended in ACI 309R.

9.3.6 Geotechnical Testing

The following testing should be considered to confirm design basis and parameters obtained at early stages during geotechnical studies.

ASTM D 6938 may be used to verify subgrade moisture content and density, and for verification of density, moisture content and relative compaction of fills.

Torvanes and Pocket Penetrometers may be used to estimate the shear strength of cohesive soils during construction. They should generally be used for comparison of relative strength since they yield only approximate shear strength data and should not be used in foundation design.

Portable static and dynamic cone penetrometers may be used to evaluate shear strength and relative density of subgrade and fill materials. They are also useful for comparison of relative subgrade strength such as in identifying relatively soft zones within foundation excavations.

Plate load testing may be performed to determine bearing capacity and settlement characteristics for shallow foundations. Correction factors should be applied to account for the footing size to be utilized since the testing is done with relatively small bearing areas.

9.3.7 Concrete Testing

Concrete testing should be performed at the job site in accordance with ACI 318 and by qualified testing technicians. At a minimum, the following should be performed:

- Obtain concrete cylinders for curing under field conditions and for subsequent testing in the laboratory
- Slump
- Temperature
- Air content (if applicable)

9.3.8 Anchor Bolt Tensioning

Anchor bolts should be post-tensioned to tension values and sequence specified by the Engineer utilizing calibrated equipment. Unless otherwise specified by the Engineer, following completion of tensioning of all bolts for a turbine, a tension check should be performed on a random 10% of the anchor bolts. Tensioning records should be kept for initial tensioning and subsequent verification testing.

10 Operations, Inspections and Structural Health Monitoring

10.1 Scope

This section addresses the post-construction inspection of the steel tower structure and foundation elements and health monitoring during the life of the structure.

10.2 Commissioning Activities

The turbine manufacturer provides to the owner design requirements for safe operation, inspection and maintenance. Upon commissioning this information includes:

- Instructions concerning commissioning for operations and maintenance
- Energization (non-structural)
- Commissioning tests
- Records including weld inspection reports, flange bolt tensioning, material certifications and warranty documents related to products incorporated into the tower structure
- A service manual and maintenance manual
- Work procedures plan
- Emergency procedures plan including OSHA guidelines for fall arrest and rescue.

Once the wind turbine tower is commissioned, the owner assumes responsibility for maintaining the structure in accordance with the commissioning documents provided by the turbine manufacturer.

10.3 Post Construction Inspections – Towers

The structural components of wind turbine towers are inspected at the minimum time intervals required by the Turbine Manufacturer and Certification Agency. Inspection time intervals will vary by component and location to account for higher corrosive environments. International Building Code (IBC), Chapter 17 Structural Tests and Special Inspections shall also apply to this section as required by the engineer. Mill, fabrication and welding certifications of the tower materials are maintained by the Turbine Manufacturer and not addressed in this section.

10.3.1 Tower Structure

The following minimum items are recommended to be inspected or monitored:

- Structural connections of climbing facilities, platforms, and other supporting systems on tower
- Bolt retensioning records
- Physical condition of bolt and nut locking devices
- Paint coatings, epoxy and/or galvanizing condition of all tower components
- Straightness of tower structure and movement of soil adjacent to foundation perimeter

Long term retensioning frequency will vary by the Tower Manufacturer and will typically be recommended in the Tower Manufacturer's service manual.

10.3.2 Bolted Connections

Bolt inspections are performed using calibrated equipment to verify that correct tension exists in bolts following their balancing, steel grade and condition of bolts.   To prevent premature degration of the structural elements at the locations of the bolted connections, an ongoing routine inspection program that includes the Tower Manufacturer is recommended. This inspection program takes into account the steel relaxation associated with bolt lengths, testing methods (ping testing, others), and connection types. The bolted connections between the Tower and a grouted base connection interface with the Foundation are particularly critical, as uplifting of the base flange during extreme weather events can result in localized overstress and crushing of the grout and excessive movement. Applicable bolting standards include the Specification for Structural Joints Using A325 or A490 Bolts by the Research Council on Structural Connections Committee A.1 (RCSC)

10.3.3 Welded Connections

The Tower Manufacturer maintains and provides records of original mill certifications of steel, ultrasonic or magnetic particle weld inspections, and other quality assurance records that document compliance with the specifications. Any new welding on the tower structure should be inspected by an AWS certified weld inspector and be performed in accordance with weld specifications outlined in AWS D1.1.

10.3.4 Corrosion Protection and Coatings

Large wind turbine support structures can be protected against the environment during the life of the structure through various technologies. Paint coatings and hot dipped galvanizing are two separate proven technologies. The US specification for galvanizing is ASTM A123 Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products, and ASTM A153 Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Hardware. Paint coatings are applied in fabrication in accordance with ISO-12944-2: 1998 Standards for corrosion protection of steel structures by protective paint systems. Damage to any coating of the tower structure is important to repair upon discovery.

It is equally important that corrosion protection of the tower base anchor bolt post-tensioning is achieved. This starts during the installation process when the post-tensioning bolts should be sealed and kept dry. A protective external coating of the exposed bolts prior the placement of the tower and tensioning is recommended.

10.4 Post construction Inspections - Foundations

Foundation elements are partially or fully buried below grade, requiring these concrete elements to be inspected both prior to, during and after their construction. This section addresses the element of the foundation above grade.

The following minimum items are recommended to be inspected or monitored:

- Visible cracks in the above-grade portion of concrete foundation
- Concrete degradation or cracking in exposed foundation
- Visible inspection of the condition of the grout for cracking of spalling
- Settlement of tower foundation or surrounding soil

Concrete cracking in the foundation elements should be examined by the engineer to establish cause. Repair of surface cracking, when not indicative of a structural concern, is recommended to repair to reduce the ingress of water and elements that can cause corrosion and reduce the designed life cycle of the foundation element.

Non-destructive testing technologies that may be required to verify the condition of the foundation when damage occurs are varied and selected based on the location on foundation, density of local reinforcing steel and site constraints.

Guidelines of concrete inspection procedures are provided by the American Concrete Institute (ACI), Post-Tensioning Institute (PTI) and the Prestressed Concrete Institute (PCI). It is recommended that the Engineer of Record (EOR) provide inspection and non-destructive testing criteria for the foundation during the service phase of the turbine's life. These inspections may include frequency of concrete and grout inspection, anchor bolt testing for tension and condition surveys to establish level of any corrosion of the structural elements. ,

10.5 Structural Health Monitoring

Sensors may be used to monitor the structural behavior of the wind turbine structure. The tower structure can be monitored for localized fault finding, tilt and vibration intensity. The primary methods of health monitoring include accelerometers, velocity meters, displacement measurement and strain gauges. Accelerometers are used to measure the dynamic response and natural frequencies of the tower and can detect signs of changes in the structure. Strain gauges and other displacement measuring devices detect highly localized changes in the structures condition. The application of strain gauges are often used in prototype towers. Sensors are not commonly used in production towers.

10.6 Life Cycle

The life cycle of a wind tower structure can be extended through preventive visual inspections at the time of its installation to assure all elements are properly installed and post construction inspections.

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12 Appendix A: Large Wind Turbine Structural Compliance Checklist

The following Checklist is offered as a recommended verification list to be used by Authorities Having Jurisdiction involved in the permitting process as it relate to wind turbine structural integrity. Statements or PE-stamped documentation should be accompanied, as needed, by Assessments (itemized below in 3 - 9). Itemized Assessments below (3 – 9) should be performed by reputable Independent Engineer(s). This Checklist can be used for entire wind farms or individual wind turbines.

___ 1. Statement or certification of wind turbine (rotor-nacelle assembly) compliance meeting at least one of the following:

- ___ 1a. Statement indicating local conditions meets those assumed for the design of wind turbine(s) as per conditions in Section 11 of IEC 61400-1.
- ___ 1b. Statement indicating structural integrity of wind turbine is not compromised under local conditions when these conditions equal or exceed those assumed in design of wind turbine. Should also fulfill Item #9a below.

___ 2. PE-stamped calculations and drawings that show design code compliance of support structure (tower and foundation) is not compromised under local conditions for wind turbine to be installed. Should also fulfill Item #9b below.

The following Independent Evaluations refer to Section 11 of IEC 61400-1 and this Guide, as may be required for a given wind farm project:

___ 3. Assessment of topographic complexity (if any)

___ 4. Assessment of wind conditions

___ 5. Assessment of wake effects from neighboring wind turbines

___ 6. Assessment of other environmental conditions

___ 7. Assessment of earthquake conditions

___ 8. Assessment of soil conditions

___ 9. Assessment of structural integrity by load calculations by Professional Engineer (if Items #1 and #2 above are not marked)

- ___ 9a. Wind turbine component verification report or certification (when Items #1b above is marked)
  - ___ Load cases or special design situations
  - ___ Load calculations
    - ___ Fatigue Loads
    - ___ Ultimate Loads
    - ___ Load carrying component buckling and deflection analyses
- ___ 9b. Support structure design documentation or verification report (when Items #2 above is marked)
  - ___ Load cases or special design situations
  - ___ Load calculations
    - ___ Fatigue Loads
    - ___ Ultimate Loads
    - ___ Load carrying component buckling and deflection analyses

13 Appendix B: Loads Document Sample Format

In wind industry standard practice, the *Loads Document* has evolved into an efficient way to communicate *WTGS* loads to *Certification Agencies* and to component designers, such as tower or foundation design engineers. While there is no industry standard for *Loads Document* report formatting, the required content of most loads documents is somewhat uniform. For this reason, where appropriate, an agency-specific presentation format may be referenced, and it should be emphasized that this *Recommended Practices* endorses no particular *Certification Agency* standards. The focus of this section is to identify specific information found in loads documents that is especially useful to perform the structural design of the *WTGS* support structure.

Recommended Content:

- Geometric Parameters including:
- Coordinate axis definitions, i.e., the X-Y-Z used as reference
- Tower hub vertical offset dimension above tower top
- Maximum permissible tower diameter at the blade tip pass elevation
- Required ring flange geometry at the turbine base-to-tower mounting interface
- Parameters for Transport and Erection Logistics:
- Maximum permissible diameter for any tower section
- Maximum permissible weight for any single tower section
- Maximum permissible length for a tower section
- Turbine Parameters:
  - Turbine mass properties including
  - Center of gravity (C.G.) coordinates of the total turbine and also of individual components such as the nacelle and rotor (hub and blades).
  - Weight of the total turbine and also of individual components such as the nacelle and rotor (hub and blades).
  - Mass moments of inertia about the turbine C.G.
  - Turbine operating frequencies (speeds), the range of operating frequencies, distinct operating frequencies (e.g., where separate high and low speed generators exist), and any other operational frequencies significant for the tower designer to avoid resonance by providing adequate separation from the *WTGS* turbine-tower-foundation system natural frequencies.
- Tower Loads (clearly designating both unfactored characteristic (i.e., “service”) loads and factored design loads) including
- Envelope of governing extreme loads
- Envelope of DLC 6.1 cases for *local building code* compliance.
- Envelope of operational load cases to be used in the earthquake load combinations.
- Fatigue loads including:
  - Fatigue Damage Equivalent Loads at the tower top, base, and preferably at several intermediate elevations such as at splice flange locations.
  - Fatigue Loads in Markov matrix format at the top flange, base plate, and intermediate splice flange elevations.
    - Fatigue Loads in Markov matrix format at other tower locations as requested by the *Engineer* where fatigue design procedures are such that damage equivalent loads are not sufficient.

Sample Format

Extreme Loads

EXTREME LOADS											
		F_x	F_y	F_z	F_{res}	M_x	M_y	M_z	M_{res}	Load Case	Load Factor
F_x	Max										
	Min										
F_y	Max										
	Min										
F_z	Max										
	Min										
F_{res}	Max										
	Min										
M_x	Max										
	Min										
M_y	Max										
	Min										
M_z	Max										
	Min										
M_{res}	Max										
	Min										

Fatigue Damage Equivalent Loads

FATIGUE DAMAGE EQUIVALENT LOADS							
Nref		F_x	F_y	F_z	M_x	M_y	M_z
m is the slope parameter of the S/N curve	m_a						
	m_b						
	m_c						
	m_d						
	m_e						
	m_f						
	m_g						
	m_h						
	m_i						
	m_j						

14 Appendix C: ASCE 7 versus IEC 61400-1 velocity profiles

ASCE 7 wind velocity profile is given by Equation C6-1. Coefficients are shown in Table below which were adjusted for a standard height of hub height and for a matching wind speed averaging time.

(Eq C6-1)

ASCE 7 Exposure Category		
Exposure D (flat facing water-like terrain)	1.232	0.111
Exposure C (open terrain with little obstructions)	1.000	0.154
Exposure B (suburban/urban)	0.693	0.250

IEC 61400-1 wind velocity profile is given by Equation C6-2.

(Eq C6-2)

α = 0.2 for normal wind conditions

α = 0.11 for extreme wind conditions

Graphic below shows that ASCE 7 velocity profile and IEC 61400 velocity profile match very well for open terrain with little or no obstructions. Terrains with Exposure D should use velocity profile from ASCE 7 modified for exposure as given by Eq C6-1.