Seismic Forces for Wind Turbine Foundations
Wind Turbine Structures, Dynamics, Loads and Control

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Purpose

For wind energy projects located in seismically active regions, considerations for seismic forces often utilize criteria developed for building structures. The loads determined from building code procedures are then superimposed with operational turbine loads for turbine foundation design. Utilizing criteria developed for seismic evaluation of building structures raises questions of applicability for other structures such as conventional wind turbines. It is therefore important to understand seismic response behavior of wind turbines in order to appropriately apply building code procedures for seismic loading evaluation. As additional wind generation capacity is added to seismically active regions, the procedures for determining and applying seismic forces to a wind turbine foundation need to be better understood in order to ensure proper consideration thereof. More refined application of seismic criteria to wind turbine foundation design could provide better-optimized foundation systems, and in turn contribute to the ability for the wind industry to grow as a reliable and competitive source of clean and renewable energy. The purpose of this paper is to look at current practice for seismic loading determination for wind turbine foundations, discuss the limitations of current design methods, and suggest some improvements to design procedures to better represent the behavior of wind turbines under seismic loading.

Abstract

Evaluation of wind turbine seismic loading typically follows wind industry design standards which require calculation of a representative horizontal seismic load using local building code procedures and superimposing the load with the turbine emergency stop or normal operational loads. This general approach is recommended by wind industry standards including IEC 64100 [4], GL Guidelines for Certification of Wind Turbines [5] and DNV/Risø guidelines [6].

Building code seismic design provisions have a main goal of providing for safe exit of building occupants in the event of a design earthquake ("life safety"). This is undertaken by requirement of redundant load paths and prescriptive structural detailing that provide mechanisms for energy dissipation. Insofar as wind turbines can be considered unoccupied structures, there is a fundamental disconnect between the goal of the building code and practical requirements for wind turbines and their foundations. Building code procedures also assume certain dynamic characteristics that are not always applicable to wind turbines. In some cases this can be shown to be both overly conservative and unconservative with some dependence on whether frequency or time domain methods are employed in evaluation of the seismic loading.

1. INTRODUCTION

Most local building codes within the U.S. are based on the requirements of the 2006 International Building Code (IBC). The seismic design provisions within the 2006 IBC are in turn based on the recommendations of ASCE 7-05 (Minimum Design Loads for

Buildings and Other Structures) and FEMA 450 (NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2003). These documents will herein be referred to as the “code” or “code documents”. The analysis procedures based on the recommendations of ASCE 7-05 and FEMA 450 are largely similar with most information and requirements cross-referenced between the two. The provisions in FEMA 450 are more comprehensive and include extended commentary on the intent and application of the recommended procedures. To a high degree, the extensive commentary of FEMA 450 is incorporated into ASCE 7-05 by reference.

Wind turbines are not directly addressed building code provisions and interpretation, and implementation, of some aspects of the recommendations can therefore be subjective. Addressing structures such as wind turbines within the code documents would ensure that design professionals utilize consistent provisions for seismic design.

2 GENERAL BUILDING CODE APPROACH

As indicated in ASCE 7-05, building code earthquake loads assume post-elastic energy dissipation in a structure. It should be noted therefore that, while the remainder of this document addresses the computation of seismic loads on wind turbines and their foundations, seismic design requirements (e.g. structural detailing provisions) should be fulfilled for structures that may in fact be governed by load combinations which do not include earthquake loads when these structures are located in seismic regions.

2.1 Analysis Procedures

Several procedures are available for evaluation of seismic loading of buildings and other structures. The analysis procedures within the building codes are generally recommended based on the occupancy category, structural characteristics and the seismic setting of the given structure. The recommended analysis procedures can generally be categorized as consisting of modal response spectrum procedures and time history analysis procedures.

Modal (response spectrum/frequency domain) analysis can be utilized to determine seismic loads on a structure by evaluating loading contribution from all relevant modes of vibration during an earthquake. The evaluation requires determination of a response spectrum (from a building code source or site-specific evaluation) that defines the spectral acceleration of a structure as a function of the structure period. Modal analysis can be implemented to account for all relevant modes of vibration of a structure but a simplified procedure is available in the code for evaluation of only the first mode of vibration. The simplified method is commonly referred to as the equivalent lateral force (ELF) procedure in which the seismic load is calculated as an equivalent horizontal base shear. The calculated base shear is then distributed to the structure being analyzed based on the mass distribution with height. The ELF procedure provides a first order estimation of the magnitude of seismic loads and can be used as a screening tool on whether more refined analyses are required. More detailed discussion of the ELF procedures is presented in subsequent sections.

Seismic loading evaluation can also be performed using time history procedures (time domain analysis). The evaluation can be accomplished by analyzing representative time histories selected from earthquake records at a given site to more precisely model the interaction of seismic forces on the foundation, tower and turbine as the earthquake occurs. Time domain analysis is a more precise evaluation procedure since representative structural characteristics can be modeled and the response evaluated at specific time intervals during the earthquake. In the case of wind turbines, the calculated seismic loading can be combined with other concurrent loads depending on the turbine operational state.

2.1.1 Modal Response Analysis

2.1.1.1 Seismic Ground Motion Parameters

Seismic analysis procedures require defining ground motion parameter values and/or earthquake acceleration time histories. The ground motion parameter values may be determined from site-specific procedures or from the generalized procedure specified in building codes as described below.

Building codes include spectral response acceleration maps developed by the United States Geological Survey that provide the required acceleration parameter values for evaluation of seismic loads for the U.S. and its territories. The maps consist of 0.2 (S_s) and 1 second (S_1) 5% damped spectral accelerations that can more accurately be obtained from the USGS program *Seismic Hazard Curves and Uniform Hazard Response Spectra* available in public domain at http://earthquake.usgs.gov. The USGS ground motion mapping program provides a convenient tool for obtaining the mapped spectral response parameters for any location based on an input zip code or coordinates (Geographic or UTM), and also includes maps which can be reviewed for independent verification. Several analysis options are available within the program based on the code being applied for the evaluation (NEHRP, Probabilistic Hazard Curves, ASCE 7, IBC, NFPA 5000).

The spectral response accelerations from the USGS maps were created assuming attenuation relationships for soft rock and therefore require correction if the subsurface conditions are different from these assumptions. Evaluation of site-specific subsurface conditions is therefore required in order to properly account for attenuation or amplification of the ground motions indicated on the USGS maps. Building codes generally require preparation of a geotechnical report which forms a basis for the design for most structures. The seismic site class based on site-specific subsurface conditions is typically provided in the geotechnical report for a given location based on the following table extracted from Chapter 20 of ASCE 7-05.

Table 1 - Seismic Site Class - ASCE 7-05

Site Class	Shear Wave Velocity	SPT N-value	Shear Strength
A. Hard Rock	> 5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very Dense Soil	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft Soil	< 600 ft/s	<15	<1,000 psf
	Sites with more than 10 feet of soil with PI>20, moisture content > 40% and shear strength < 500 psf		
F. Soils requiring site specific response analysis	Liquefiable soils, sensitive clays, collapsible soils, peats/highly organic soils, high plasticity soils, very thick soft to medium clays		

Adjustment of the mapped spectral accelerations is accomplished by applying representative site coefficients (F_a and F_v) that effectively scale the spectral response accelerations for the appropriate subsurface conditions. The site coefficients are a function of the mapped spectral response parameter values as well as the site class as summarized in the following tables also extracted from ASCE 7-05.

Table 2 - Site Coefficient F_a

Site Class	Mapped MCE Spectral Response Acceleration Parameter at 0.2 Second Period[a]				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	___[b]	___[b]	___[b]	___[b]	___[b]

[a] Use straight line interpolation for intermediate values of S_S.
[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Table 3 - Site Coefficient F_v

Site Class	Mapped MCE Spectral Response Acceleration Parameter at 1 Second Period[a]				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	___[b]	___[b]	___[b]	___[b]	___[b]

[a] Use straight line interpolation for intermediate values of S_1.
[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

The site coefficients are utilized in determining the Maximum Considered Earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) by multiplying the mapped spectral accelerations by the respective site coefficients.

Once the MCE spectral acceleration parameter values have been determined, the design spectral response parameters (S_{DS} and S_{D1}) can be calculated and, if required, the response spectrum can be defined through simple procedures outlined in the code.

The following summarizes the steps for developing the seismic design parameters which may be used where site specific response analysis has not been performed.

1. Determine mapped MCE at 0.2 second periods (S_s) and 1 second period (S_1)
2. Determine site class based on the site-specific subsurface conditions.
3. Determine site coefficient (F_a and F_v) values and use the coefficients to calculate the adjusted MCE spectral response parameters for short periods (S_{MS}) and 1 second period (S_{M1})

$$S_{MS} = S_s F_a \quad (1)$$

$$S_{M1} = S_1 F_v \quad (2)$$

4 Determine design spectral response parameter values for short periods (S_{DS}) and 1 second period (S_{D1}) by reducing the MCE parameter values by ⅔.

$$S_{DS} = \tfrac{2}{3} S_{MS} \quad (3)$$

$$S_{D1} = \tfrac{2}{3} S_{M1} \quad (4)$$

The design spectral response parameter values can then be utilized to develop the response spectrum by following the procedures outlined is Chapter 11.4 of ASCE 7-05.

2.1.1.2 Response Spectrum

The design response spectrum is dependent on the mapped ground motion parameter values (i.e. seismic setting of the site) and the seismic site class (subsurface conditions), and is plotted as a function of the structure period. The subsurface conditions at a given site are an important part of the seismic load evaluation since the level of attenuation/amplification of the ground motion is dependent on the soil or bedrock characteristics. It is therefore critical that the subsurface conditions at a given location are well-defined in order to develop a representative response spectrum for any site.

The influence of subsurface conditions on the response spectrum is illustrated in Figure 1. The graph was developed using mapped spectral response parameters for a highly seismic area in Palm Desert, California. The resulting spectra for different assumed site classes clearly demonstrate the importance of utilizing representative subsurface conditions for a given location.

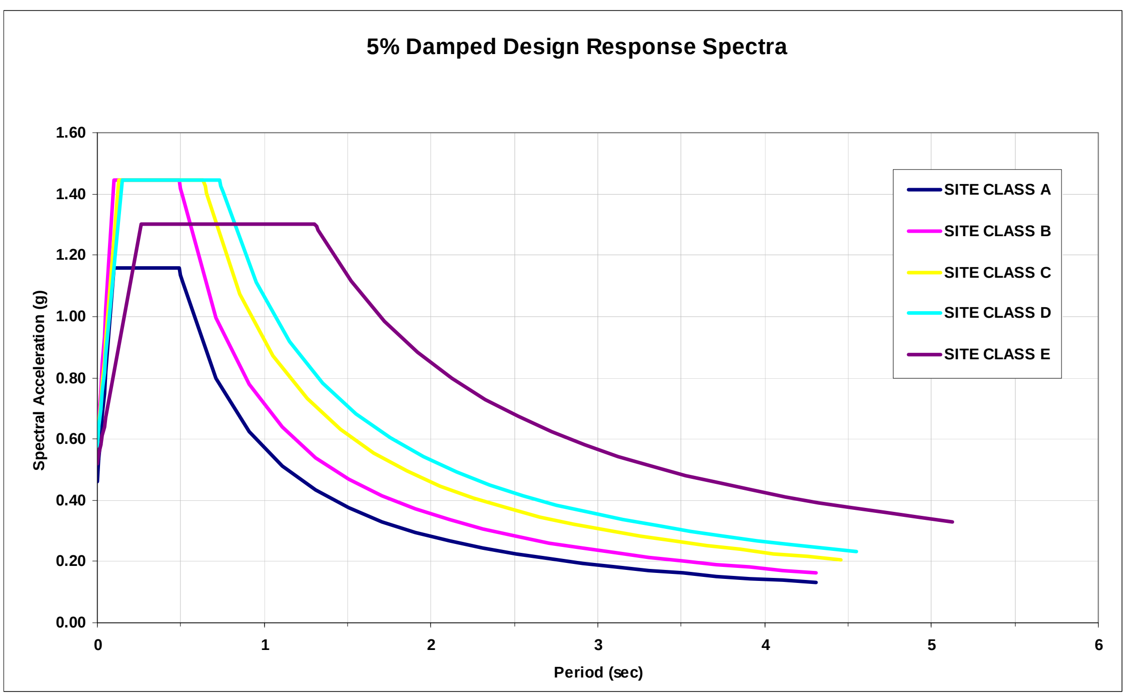

Figure 1 - Site Class Influence on Response Spectra

Once a response spectrum is developed for a given site, the dynamic response of a structure can be evaluated based on the period of vibration of the mode(s) being considered.

2.1.1.3 Structure Period

US building codes require that a structure's fundamental period utilized in seismic evaluation be based on properly substantiated analyses. An approximate formula for calculating the fundamental period of a building based on the height and structural system is presented in the code as:

$$T_a=C_t.h^x \tag{5}$$

Where h is the height of the structure and C_t and x are constants based on the structural system as indicated in Table 4.

Table 4 – Building Period Metric Constants

Structural System	C_t	x
Steel moment-resisting frames	0.0724	0.8
Concrete moment-resisting frames	0.0466	0.9
Steel eccentrically braced frames	0.0731	0.75
All other structural systems	0.0488	0.75

It is clear from reviewing the structural system categories that none of the above are a good match for wind turbines and, if the formula was to be applied, turbines would fall into the "all other structural systems" category. However, the building code indicates that Equation 5 is not recommended for non-building structures per section 15.4.4 of ASCE 7-05.

Prowell and Veers [10], conducted a study of published first fundamental mode periods for various wind turbines with differing hub heights and correlated the data with Equation 5. The resulting constants from their study representing lower and upper bound estimates of turbine first fundamental vibration period variation with hub height are presented in the following table.

Table 5 – Turbine Period Metric Constants (Data from Prowell and Veers, 2009)

Period Estimate	C_t	x
Lower Bound	0.022	1.05
Upper Bound	0.015	1.183

A graphical representation of this data is presented in Figure 2 with a band representing the Prowell and Veers upper and lower bounds compared to the building code estimate for a structure of the same height. The graph clearly demonstrates that the building code formula constants should not be used in evaluating the period of a wind turbine. A first-order estimation of wind turbine period may be obtained using Equation 5, but with the constants indicated by Prowell and Veers (Table 5). The representative periods for the relevant vibration modes of wind turbines can be calculated through direct solution of the eigenvalue problem or may be provided by turbine manufacturers.

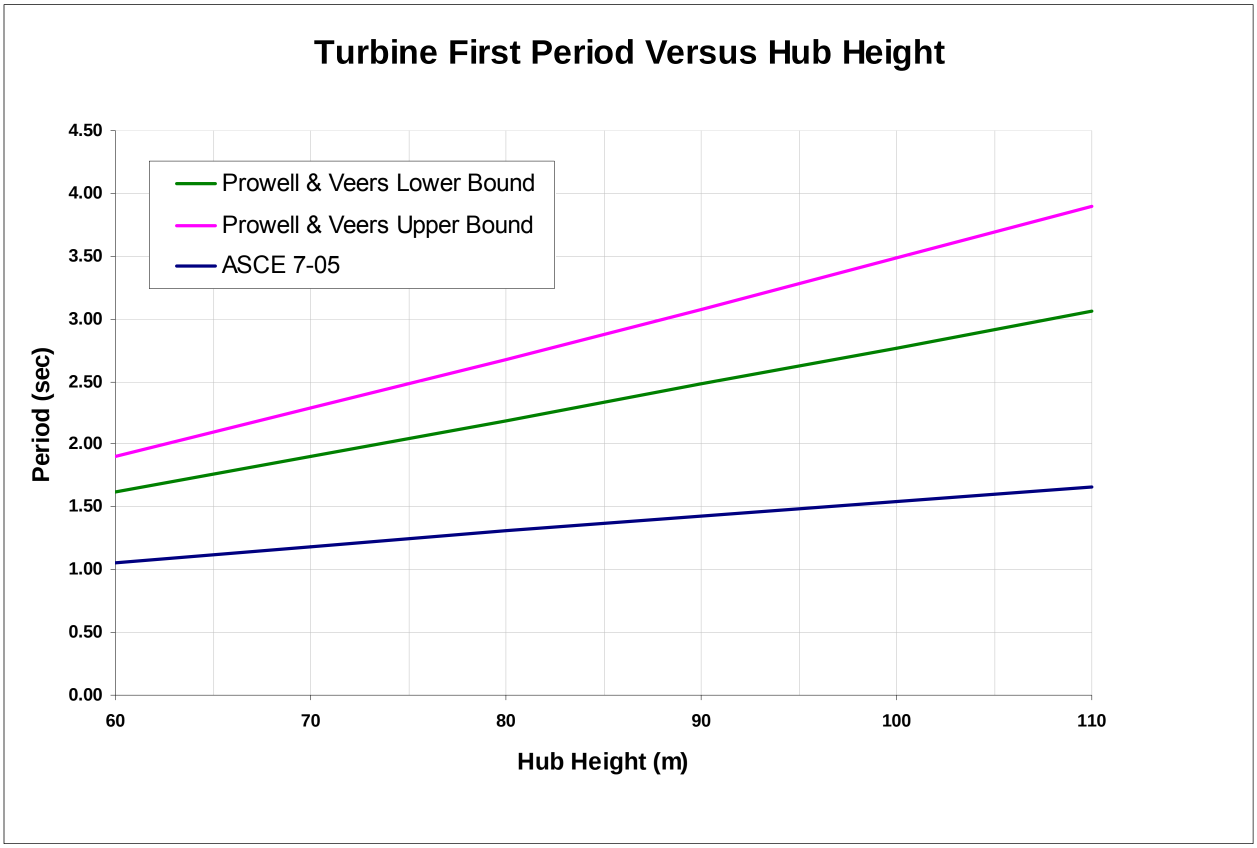

Figure 2 – Turbine and Building Period Comparison

2.1.2 Equivalent Lateral Force Procedure

Vibration characteristics of a structure can be evaluated by constructing a linear mathematical model taking into account all relevant modes of vibration each with their own characteristic modal mass, frequency and damping. The vibration characteristics can be modeled in the frequency domain and combined using appropriate methods to obtain representative response for the entire structure.

The building codes include specifications for implementing simplified modal analysis using the Equivalent Lateral Force (ELF) Procedure. The ELF procedure is effectively an application of modal vibration analysis but limited to the first mode of vibration (i.e. assumes all the structure's mass is mobilized in the first vibration mode). The ELF procedure consists of applying an equivalent static lateral force to a linear mathematical model of a structure with magnitudes and direction representative of the dynamic loading from earthquakes. The structure is assumed to be fixed at the base for application of the ELF procedure.

The total seismic force applied to a structure in the ELF procedure is calculated in terms of a base shear. The seismic base shear is calculated as the product of a site-specific seismic response coefficient and the seismic weight of the structure. The seismic response coefficient is based on the design short period spectral acceleration (S_{DS}) adjusted by a structure response modification factor (R) and an importance factor (I). The calculated base shear can then be distributed over the height of the structure in consideration of the story weights and heights as a representative model of the equivalent floor level forces from earthquake loading. The basic seismic base shear calculation formula is as follows:

$$V= C_s W \tag{6}$$

Where

$$C_S := \frac{S_{DS}}{\left(\frac{R}{I}\right)} \tag{7}$$

V=Seismic Base Shear
W=Effective Seismic Weight
C_s=Seismic response Coefficient
R=Response Modification Factor
I=Importance Factor

The seismic response coefficient can be adjusted to account for the structure period and has upper and lower bound limits depending on the seismic setting of the structure being considered. The reader is referred to the building code documents for additional details and recommended adjustments to the seismic response coefficient.

2.1.2.1 Response Modification Factor (R)

The R factor is an empirical reduction factor that is intended to account for damping, overstrength and ductility in a structural system for displacements approaching the ultimate displacement of the structure. The R factor for brittle structures with very low damping would therefore be close to about 1, which represents no reduction in the linear response of the structure. Ductile systems with significant inherent damping would conversely be able to withstand relatively large deformations in excess of the yield point and are therefore assigned a larger reduction factor (up to 8 for special moment resisting frames).

2.1.2.2 Importance Factor (I)

Each structure is assigned an importance factor (I) based on the occupancy category. The importance factor relates a structure's occupancy to hazard to human life and economic impact in the event of failure, and/or emergency response requirements. Low risk structures (e.g. agricultural facilities) are assigned an importance factor of 1.0 while structures such as residential and office buildings, schools, churches and power stations that are deemed to represent a substantial hazard to human life in the event of failure are assigned a value of 1.25. Structures that are designated essential structures which include facilities required for emergency response following an earthquake (e.g. fire stations, hospitals with emergency rooms, air traffic control towers) are assigned the maximum importance factor of 1.5.

The value of I selected for a structure impacts the calculated seismic base shear since it effectively reduces the response modification factor (ductility), thereby increasing the computed base shear if a value other than 1.0 is selected. The R/I ratio in the seismic response coefficient equation indicated above is therefore an important factor as it can increase, or reduce, the computed seismic loading demand based on the parameter values selected.

2.1.2.3 Application to Wind Turbines

Utilizing the ELF procedure for evaluating wind turbine seismic loading requires determination of representative values of the response modification factor and importance factor. It is important to recognize that while the ELF procedure may provide a reasonable first order estimate of seismic loading for most seismic settings, other modes of vibration do typically exist and can dominate behavior of the wind turbine structure for some seismic settings and should therefore be accounted for accordingly.

When utilizing the ELF procedure to evaluate seismic loading of wind turbines, it is important to note that the calculated base shear is based on 5% damped spectral accelerations. As discussed later, this level of damping is not always representative of wind turbines. Implementation of the ELF procedure also requires selection of characteristic response modification and importance factors for a given structure. Given that wind turbine are not directly addressed in building codes, selection of representative factors for wind turbines is not straightforward. The list of response modification factors based on structural systems includes a designation of R for inverted pendulum structures, which may represent a category similar to wind turbine structures. The inverted pendulum designation is described as:

Structure in which more that 50 percent of the structure's mass is concentrated at the top of a slender, cantilevered structure and in which stability of the mass at the top of the structure relies on rotational restraint to the top of the cantilevered element (ASCE 7-05 Chapter 11)

Since wind turbines typically have 40 to 50 percent of their mass concentrated at hub height, they do not completely match this description with regard to the mass distribution. However, it appears that it is the closest designation to which a wind turbine can be assigned among the categories indicated in the code. The response modification factor assigned to inverted pendulum structures is R=2.

Once the appropriate response modification and importance factors have been selected, the seismic base shear can be evaluated based on the procedures outlined above, using the weight of the turbine typically provided in a manufacturer's foundation loading document. Assuming an importance factor of 1 is selected, the R/I ratio would then be 2 based on the R for an inverted pendulum as indicated above. In the case of "essential" structures designated as Occupancy Category IV, ASCE 7 Section 15.7.10.5 indicates that for shell structures where buckling (local or general) may be the primary mode of failure, the ratio R/I should be limited to 1. This in effect discounts any redundancy or ductility of the tower and would have the effect of doubling the calculated seismic demand for a given structure, all other things being equal. Therefore, evaluation of the wind turbine seismic loading should include assessment of the primary mode of failure of the tower. It is also clear that designation of a representative occupancy category is required for wind turbines in order to assure consistent application of the seismic provisions across the design community.

One important point in this discussion is whether a wind turbine generator is considered an essential structure as defined by the building code. Typically, conventional power plants, and some single pedestal water towers have been considered essential structures and assigned to Occupancy Category IV as defined by the 2006 International Building Code. Wind turbine generators have historically not been assigned to the "Essential Structures" category. As wind energy grows as a percentage of our national energy supply, the relative importance of wind turbine generators may need to be re-evaluated, and the design category changed accordingly.

2.1.2.4 Soil-Structure Interaction Effects

As previously noted, the ELF procedure assumes a structure with a fixed base and inherently discounts any influence of the mass, stiffness, or damping of the foundation system. The case of a perfectly rigid foundation is rarely encountered in practice as recognized in the code commentary.

Seismic analysis procedures can be augmented to include the effects of soil-structure interaction by recognizing that, for structures on elastic soils, the foundation response would be different from free-field response. For situations where the structural design does not include the effects of foundation flexibility, mass, and damping, building code procedures include recommendations for calculating soil-structure interaction effects in evaluation of seismic loading. Reduction in calculated base shear of up to 30 percent is permitted based on the recommendations of Chapter 19 of ASCE 7-05. Input parameters for the analysis include foundation horizontal and rotational stiffness, effective damping of the foundation-structure system and fundamental period of the structure. The outlined procedures recommend evaluation of seismic response utilizing appropriately modeled soil stiffness values based on the strain levels associated with the design earthquake. The code also recommends using subgrade stiffness variations between 50% and 150% of the design stiffness in order to account for variations that may not be captured during a regular geotechnical investigation.

Soil-structure interaction effects should be accounted for in seismic loading evaluation on wind turbines as they are likely to indicate an increase of the structure fundamental period and an increase in damping which may impose lower seismic demand when compared to a fixed-base structure.

2.1.3 Time History Procedures

Seismic loading on a structure can also be evaluated in the time domain using earthquake acceleration time histories. Linear and non-linear time history analyses can be performed but linear response history procedures are adequate for wind turbines per industry standards.

Representative synthetic time histories or actual time histories selected from earthquake records at a given site can be utilized in performing time domain analysis. For most locations within the US, earthquake acceleration time histories are available in public domain from sources such as the California Strong Motion Instrumentation Program (CSMIP) at the California Geological Survey.

3 NUMERICAL MODELING

Evaluation of the various modes of vibration of a structure typically requires the use of a computer program for processing the input data and performing the computations required to solve the equations of motion. Modal analysis of wind turbine seismic loading can be performed using software such as GH Bladed, an industry standard software package by Garrad Hassan and Partners used in the design and certification of wind turbines. In GH Bladed, the analysis is accomplished through an iterative procedure to compute the response spectrum for a reference motion, calibrating the new response spectrum against the target spectrum, scaling the spectrum in the frequency domain, and iterating the procedure as necessary (Witcher, 2004). Figure 4 shows the results of a response spectrum generated in GH Bladed utilizing a synthetic accelerogram that is calibrated against a typical building code-derived target spectrum with a good match obtained.

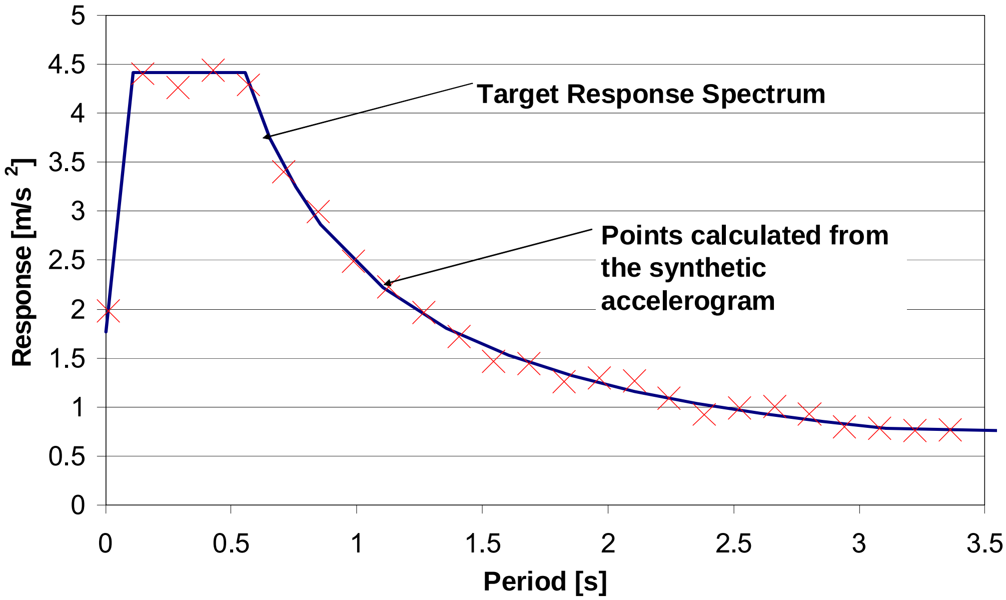

Figure 3 – Synthetic Response Spectrum from Building Code Envelope [from Witcher, 2004]

3.1 Operational State and Damping

Witcher (2004) simulated turbine seismic response under continuous operation, emergency shutdown and parked conditions with maximum tower base moments calculated with Eurocode 8 procedures within GH Bladed [7]. For the continuous operation condition, the seismic event was modeled by imposing the seismic base motion on an operating turbine without causing a shutdown. The emergency shutdown condition was modeled in a similar fashion to the continuous operation condition, except that the vibrations within the turbine were allowed to trigger an emergency stop. Aeroelastic damping under operational conditions was indicated to be approximately 5 percent (of critical damping), similar to the structural damping value assumed in current building codes. For the parked condition, the turbine was considered with only the structural damping characteristics of the turbine (i.e. without aerodynamic damping) and without additional wind loading. (Although not discussed in the Witcher paper, structural damping for a parked turbine is understood to be less that 1 percent.) Witcher's simulations considered a rigid foundation condition, and therefore did not include any influence of soil-structure interaction on the seismic response. The maximum peak-to-peak nacelle deflections interpolated from Witcher's results are summarized in Table 6. The difference in the maximum peak-to-peak deflections under power production and parked conditions was interpreted as being due to the different damping levels under the respective states. The results demonstrate that aerodynamic damping plays a significant role in the dynamic response of a wind turbine and should therefore be accounted for appropriately.

Table 6 – Nacelle Deflection under Different Operation States (Data from Witcher, 2004)

Operation Condition	Max Peak-to Peak Nacelle Deflection (m)
Power Production	0.15
Parked	0.25 – 0.3

3.2 Comparison of Building Code Procedure and Time Domain Analyses

Witcher also conducted simulations of wind turbine seismic demand using frequency and time domain methods in order to compare the results from the two methods. The results indicated that peak loads calculated using the building code (frequency domain) procedure and the time domain computations were a reasonably good match during turbine operation, even with an emergency stop during the seismic event. However when the turbine was in a parked condition, the GH Bladed simulations showed that there was a significant difference in peak loads between the building code based (frequency domain) and time domain analysis as summarized in the following tables.

Table 7a – Overall Tower Base Moment – Operational Case [from Witcher, 2004]

Frequency domain (kN m)	Time domain (kN m)	Increase (%)
29016	29866	2·9

Table 7b – Overall Tower Base Moment – Parked Case [from Witcher, 2004]

Frequency domain (kN m)	Time domain (kN m)	Increase (%)
14771	26519	79

The time domain analysis indicated an almost 80 percent increase in peak loads over the calculated building code values in the parked case. Witcher concluded that the significant difference was due to the absence of aeroelastic damping in the parked condition. It is important to note that while the difference in the calculated peak loads appears significant, it could still be the case that the seismic load case would not control the design of the turbine and the foundation since the extreme wind loads may be higher than the seismic load case.

4 FULL SCALE TESTING

4.1 Shake Table Testing

In November 2004, the University of California, San Diego Structural Engineering Department conducted the first full-scale seismic testing of a wind turbine with a 22.6 m high 65 kW turbine on the NEES Large High Performance Outdoor Shake Table. The shake table simulates earthquake motion uniaxially with a peak horizontal velocity of 1.8 m/s [9].

Figure 4 – 22.6m Hub Height Turbine Mounted on UCSD/NEES Shake Table [from Prowell, Veletzos, Elgamal, 2008]

In order to evaluate seismic performance of the turbine, excitation for the shake table consisted of the Desert Hot Springs East-West Component from the 28 June 1992 Landers earthquake (deep alluvium site, 0.15g peak acceleration, moment magnitude M_w = 7.3). The record was filtered with high pass and low pass filters to remove offset and high frequency noise. The test was conducted in the linear elastic range, with the earthquake record scaled at 50%, 100%, 143%, and 200%. For safety reasons, the test was conducted with the turbine in a parked condition.

The shake table testing program found that there was significant amplification of the input seismic acceleration in the nacelle during all shake table tests of the turbine. Damping values were also evaluated during the test to estimate the magnitude of structural damping at the first natural frequency using log decrement and half power methods. A summary of the testing results from Prowell, Veletzos, Elgamal, 2008 [9], is presented in Table 8.

Table 8 –Full Scale Test Nacelle Accelerations and Tower Damping

Input Motion	Max Acceleration (g)		Damping (%)	
	Input	Response	Log Decrement	Half Power
50% Landers	0.07	0.19	2.00	0.60
100% Landers	0.12	0.28	0.86	0.64
143% Landers	0.17	0.52	0.43	0.66
200% Landers	0.24	0.70	0.41	0.52

As with the simulations by Witcher, Prowell et al. concluded that aeroelastic damping has a significant effect on the response of the turbine system, which is not accommodated in common building code approaches. Prowell also suggests that aeroelastic damping is directional, and would not always coincide with the direction of seismic shaking.

A direct comparison with building code values was not conducted as part of the UCSD research. However, the low damping values indicated in the UCSD testing support the conclusions by Witcher as they indicate relatively low damping values in the parked condition. Based on the work by Witcher, there appears to be good agreement between building code values and peak seismic loading under operational conditions, both when the turbine continues to operate through the earthquake and when the earthquake triggers an emergency stop. This is primarily due to the presence of aeroelastic damping while the blades are spinning. Conversely, if an earthquake strikes while the turbine is parked, the lack of aeroelastic damping can lead to a significant amplification of the earthquake loads on the turbine.

5 CONCLUSIONS

Based on the above review, commonly utilized building codes do not appear to consider wind turbine systems in a truly comprehensive manner. Several conclusions can be drawn with regard to current practices and understanding of wind turbine seismic loading as summarized below:

- Current practice in assigning importance factors and occupancy categories to wind turbine generators is not consistent with similar structures such as single pedestal water towers, or of similar importance to the electrical grid, such as a conventional power plant.
- Combinations of loads prescribed by the IEC and other standards appear appropriate provided that aeroelastic damping is present.
- If aeroelastic damping is not present (i.e. a parked condition), standard building code procedures do not allow for an adjustment in damping ratios different from those observed in conventional building systems, and therefore cannot take the low level of damping of a parked turbine into consideration. More refined analysis would need to be conducted to take the lower damping ratios into account.
- Building code procedures do not account for directivity of seismic loading and may not predict representative loading if the direction of earthquake loading is not parallel to the wind loading direction, thus potentially skewing the seismic + wind load combinations.

The simulations by Witcher suggest that the level of damping for a wind turbine structure during operation, or shutdown, is comparable to typical building code assumed damping levels of about 5%. However, the UCSD testing clearly indicates that the 5% level of damping is not representative of a wind turbine in parked conditions where the turbine damping levels are significantly lower. The low level of damping indicated by the UCSD testing supports Witcher's conclusion regarding the simulation results depicted in Tables 6, 7a and 7b. It should therefore be recognized that time domain analyses rather than frequency domain procedures are better suited for evaluation of wind turbine seismic loading under parked conditions.

6 FUTURE RESEARCH NEEDS

In order to better understand the seismic response of a wind turbine, several aspects require additional research and validation:

- Further measurement and testing of damping of the wind turbine system under parked and operational conditions.
- An investigation of directivity of aeroelastic damping and seismic excitation to determine possible implications on wind turbine vibration.
- Impact of soil-structure interaction in the seismic response of the foundation and wind turbine.
- A determination as to whether wind turbine systems are considered "essential structures" by the building code, and validation of the appropriate building code Response Modification Factor for wind turbines.
- Probabilistic analysis of the interaction between extreme wind and seismic events
- Evaluation of post buckling behavior of tube towers.

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