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Assessment of Wind Turbine Seismic Risk: Existing Literature and Simple Study of Tower Moment Demand

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Assessment of Wind Turbine Seismic Risk: Existing Literature and Simple Study of Tower Moment Demand

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Abstract

Various sources of risk exist for all civil structures, one of which is seismic risk. As structures change in scale, the magnitude of seismic risk changes relative to risk from other sources. This paper presents an introduction to seismic hazard as applied to wind turbine structures. The existing design methods and research regarding seismic risk for wind turbines is then summarized. Finally a preliminary assessment is made based on current guidelines to understand how tower moment demand scales as rated power increases. Potential areas of uncertainty in the application of the current guidelines are summarized.

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1 Introduction

Earthquakes of varying magnitude are regularly experienced throughout the world. Most earthquakes are small and cannot be perceived without special instrumentation. Large earthquakes are rare, but can present a significant risk of damage to civil structures. The average time between large earthquakes (the return period) is often measured in hundreds or thousands of years. With a design life of approximately 20 years most turbines will not experience a strong earthquake, but as with all civil structure the wind turbine is subjected to some level of seismic risk. Wind energy developers in earthquake-prone regions can mitigate this risk with appropriate design margins, site selection, or insurance. It is important to understand the characteristics of the seismic risk in order to make an informed decision on what approach to take.

The probability of damage from an earthquake (seismic risk) can be conditioned into two separate probabilities. The first, seismic hazard is the probability that an earthquake with particular characteristics, such as peak ground acceleration (PGA) above 0.5 g, will occur at the site where the turbine is installed. Seismic hazard is introduced and summarized but no original insight is provided. The second component is the likelihood that damage will occur conditioned on the occurrence of an earthquake with particular characteristics. This investigation focuses on existing techniques and published research for assessing the impact of earthquake loading on design loads for wind turbines.

Existing literature and a preliminary assessment are presented to clarify specifics of seismic risk as they pertain to wind energy. The literature review presents a summary of the requirements imposed by turbine specific guidelines for consideration of seismic loading. Next, a review of published modeling techniques for seismic loading of turbines is presented. Following this a summary of the published findings specific to seismic loading of turbines is presented. A brief discussion of perceived and actual risk is then presented. This investigation closes with a preliminary assessment of tower moment demand. The assessment provides a high-level overview of pertinent considerations for seismic loading.

2 Earthquake Risk Assessment

2.1 Measuring Earthquakes

There are a number of different methods available for characterizing an earthquake. The magnitude of an earthquake is a measure of the amount of energy released. Two common measures of earthquake magnitude are Richter Magnitude and Moment Magnitude. Richter Magnitude is defined by the response of a Wood-Anderson seismometer located 100 kilometers from the epicenter of the earthquake. The Wood Anderson seismometer cannot measure earthquakes larger than approximately 6.8 due to physical limitations of the instrument. The Moment Magnitude scale overcomes this limitation by defining the magnitude in terms of physical quantities, including the size of the rupture surface, that do not depend on the instrument used for measurement. The magnitude of a particular earthquake is a constant and does not dependent on the location.

It is also important to quantify the impact of an earthquake at a given location. The local effect can be measured using the Mercalli Magnitude Scale, which is a measure of earthquake intensity at the site where the earthquake is observed. The Mercalli Magnitude Scale converts qualitative measures of an earthquake to a quantitative measure. Another local measure of an earthquake is the PGA, which is the peak acceleration observed at a site on the ground surface. Both the Mercalli Magnitude and PGA of an earthquake will generally decrease as distance from the earthquake increases.

2.2 Seismic Hazard

The first component of determining seismic risk is quantification of seismic hazard. A common method of quantifying the hazard is to conduct a probabilistic seismic hazard assessment (PSHA). A PSHA consists of three major steps: define the seismic sources that impact the location of interests; define the characteristics of each seismic source; and probabilistically combine the characteristics of each seismic source to obtain a probability distribution of a site-specific earthquake measurements such as PGA. Figure 1 graphically shows the results of a PSHA conducted in 2002 that calculated the acceleration that has a 10% probability of being exceeded in 50 year throughout California for firm rock. For comparison the acceleration that has a 10% probability of being exceeded in 50 years is shown in Figure 2 and Figure 3 for Asia and Europe, respectively. These three maps of seismic hazard are produced by different agencies and are not on the same scale. Both maps for Europe and Asia show a maximum acceleration of approximately 0.5 g, but the map of California shows a maximum of 0.8 g.

2.3 Return Period

As previously mentioned, seismic hazard is often described using statements such as, "There is a 10% probability that the PGA will exceed 0.5 g in 50 years." This statement is equivalent to saying that there is a 90% probability that no earthquake large enough to cause a ground acceleration greater than 0.5 g will occur in 50 years. To justify such statements a statistical model of earthquake occurrence must be selected. The occurrence of a large earthquake capable of exceeding a specified PGA is frequently modeled as a homogeneous Poisson process (HPP). The application of the HPP process embeds three assumptions:

1. The probability of an event occurrence in a given time interval is approximately proportional to the length of the interval.
2. The probability of more than one event occurrence in a short time interval is negligible compared the probability of one event occurring.
3. That event occurrence in non-overlapping intervals is statistically independent.

The HPP is not strictly accurate for modeling earthquake occurrence. The third assumption states that no knowledge of the process history is needed to calculate probability for a given time period. Earthquake occurrence is a complicated physical process that has memory. For example if a large earthquake occurs the probability of another earthquake, such as an aftershock, occurring is elevated. Despite this discrepancy the HPP provides useful results for quantifying the statistics of earthquake occurrence. Given a period of time, *t*, and a mean arrival rate, λ, the probability, *P(t)*, that an earthquake capable of exceeding the given PGA occurring is

$$P(t) = 1 - P_0(t) = 1 - e^{-\lambda t} \approx \lambda t - \frac{\lambda^2 t^2}{2} + \frac{\lambda^3 t^3}{6} \ldots$$

where $P_0(t)$ the probability that no earthquakes will exceed the given PGA in time *t* and the mean arrival rate, λ, is the inverse of the return period. For example, a 475 year return period is equivalent to a 10% probability of occurrence in 50 years. If the timeframe is shortened to 20 years an event with a 475 year return period has a 4% probability of occurrence.

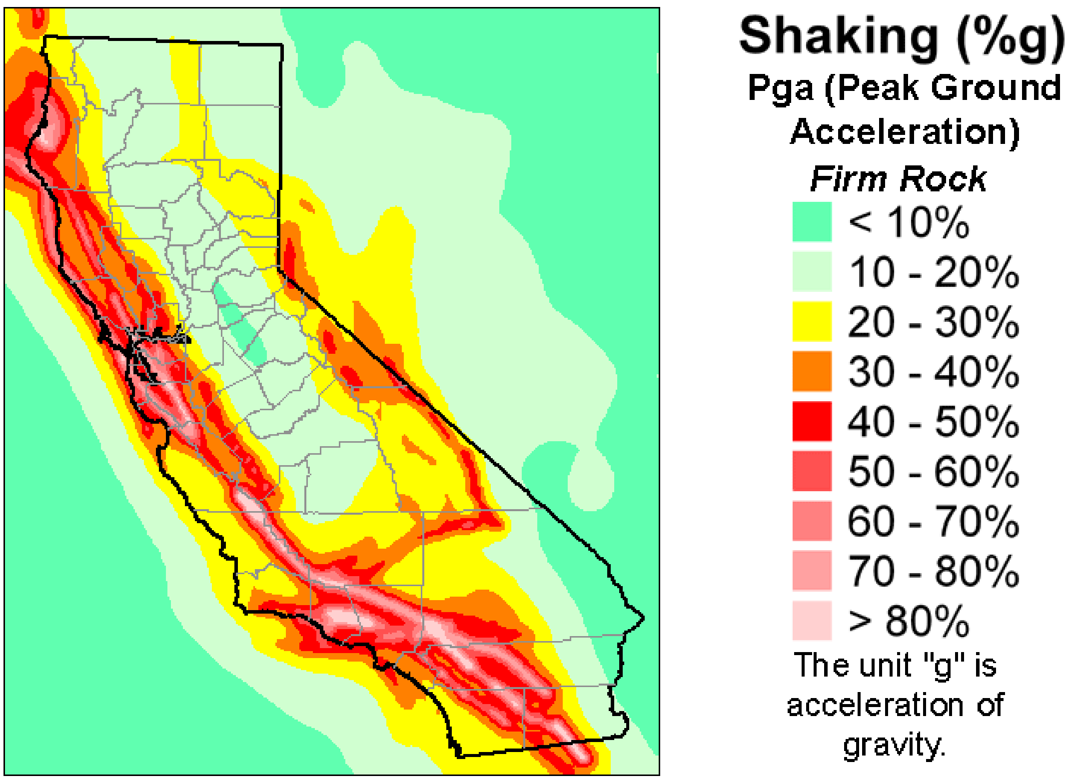

Figure 1: Seismic Hazards in California (Source: California Geologic Survey)

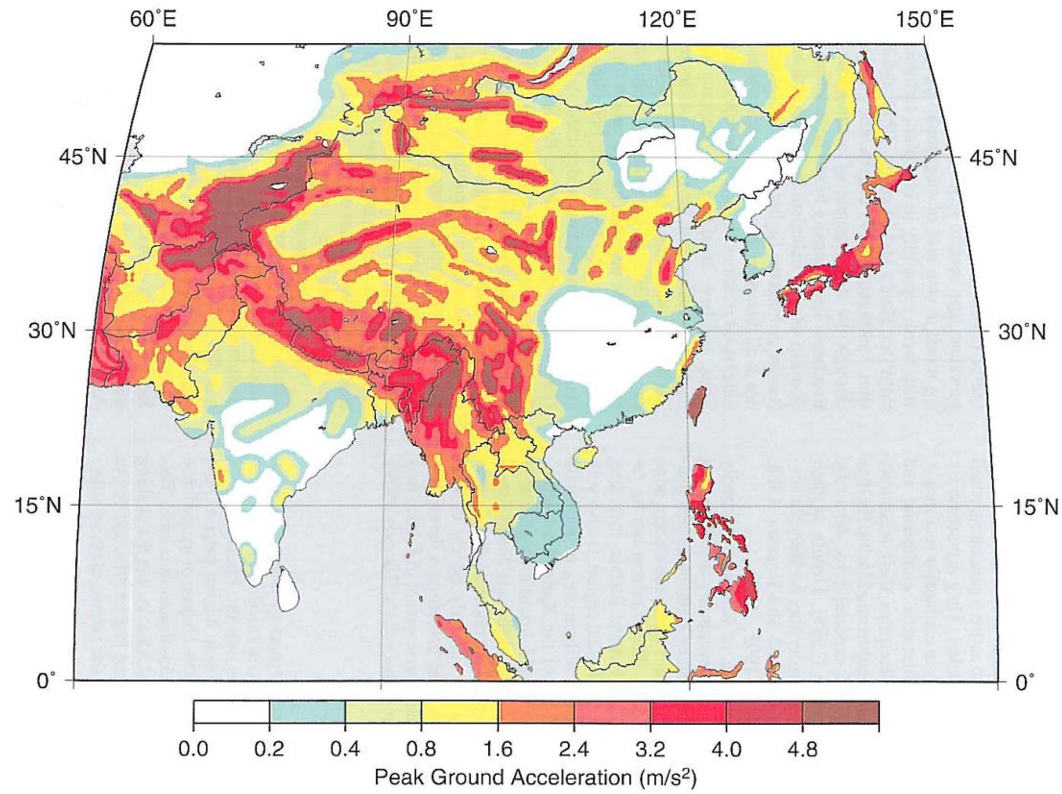

Figure 2: Seismic Hazards in Asia (Zhang et al., 1999)

2.4 Response Spectrum

The response spectrum is a tool used to evaluate the response of a structure to seismic loading. An elastic response spectrum (ERS) shows dependence of the maximum response acceleration of a single degree of freedom (SDOF) structure with a prescribed damping when subjected to a particular excitation, generally an acceleration time history. Figure 4 shows the 5% damped ERS calculated by numerical integration of some recorded earthquake acceleration time histories.

As seen in Figure 4 the resulting acceleration for a particular period and damping can vary greatly depending on the time history. Building codes, such as the International Building Code 2006 (IBC) (ICC, 2006), incorporate seismic hazard and structural properties to create what is called a design response spectrum. A design response spectrum provides a unique mapping from period to spectral acceleration.

The procedure for constructing a design response spectrum varies depending on the applicable building code, currently the IBC 2006 in many parts of the United States. The IBC 2006 assumed a damping of 5% of critical. A probabilistic seismic hazard assessment was conducted to determine the spectral response acceleration with a 2% probability of exceedance in 50 years for structures with periods of 0.2 seconds and 1 second (ASCE, 2006). The resulting spectral response accelerations and local soil conditions are used to determine site specific modification factors from lookup tables. The original spectral response accelerations are multiplied by the site specific modification factors to account for local soil effects. After being adjusted the spectral response accelerations are multiplied by two thirds to reduce the conservatism associated with a

2% probability of exceedance in 50 years to calculate the design response accelerations. These two design response accelerations are scaled as a function of period to build a complete design response spectrum. An example IBC 2006 design response spectrum is shown in Figure 5. The designer can quickly obtain a single design response acceleration from the design response spectrum for seismic analysis. This single number represents the acceptable risk prescribed by building code.

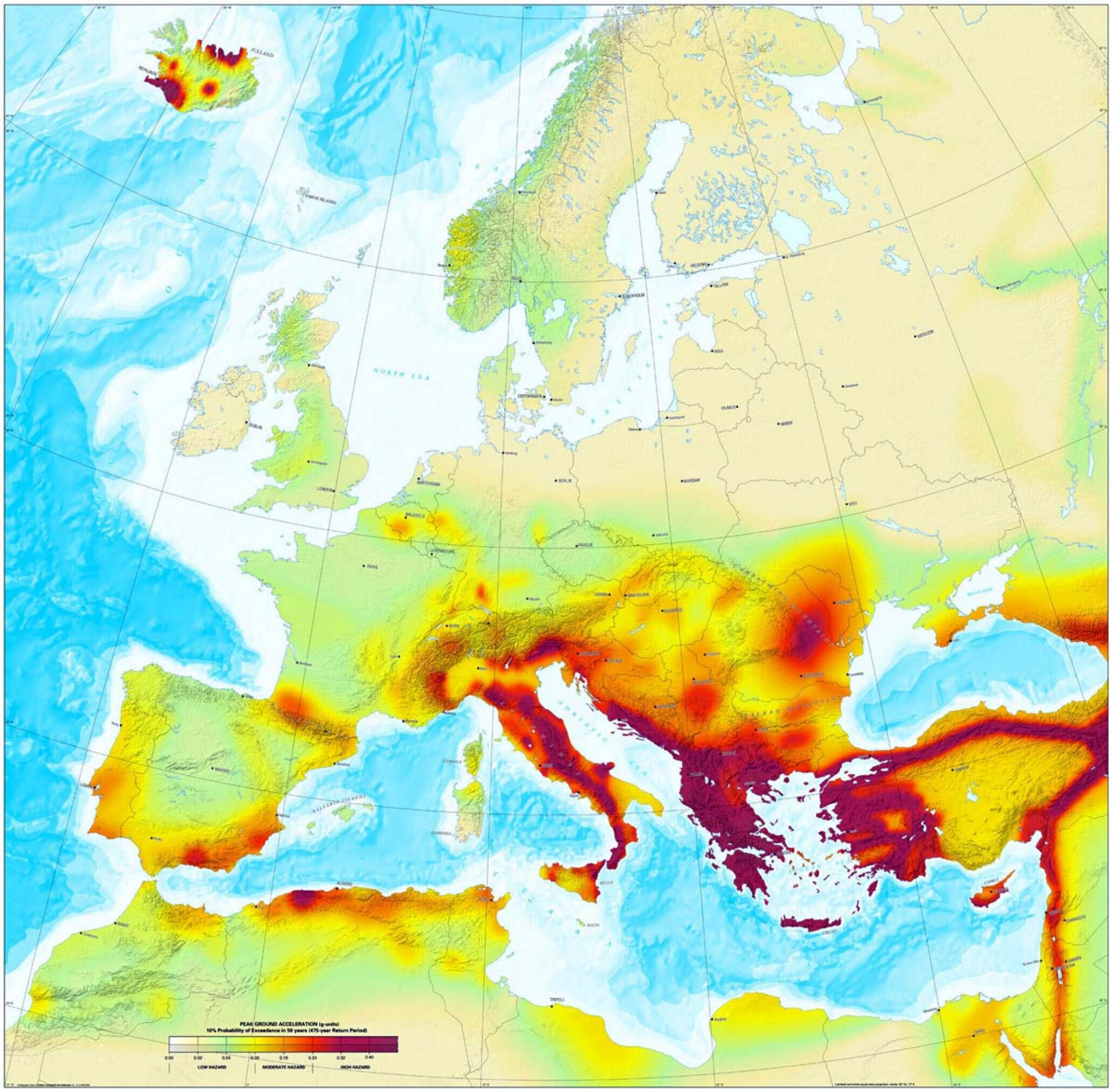

Figure 3: Seismic Hazards in Europe (Source: ESC-SESAME)

2.5 Preliminary Structural Risk Assessment

The second component of evaluating seismic risk is to evaluate the earthquake response of a structure. Three main factors are used to conduct a preliminary assessment of structural response to seismic loading: first natural period; damping; and ductility of a structure.

A structure is characterized by a set of orthogonal modes shapes which serve as a basis to describe vibration of that structure. Each of the mode shapes of a structure will occur at a given period. The first natural period is defined as the longest period at which a mode shape occurs. A preliminary seismic risk assessment only considers vibration in the first mode shape at the first natural period. It is common for buildings codes to assess seismic risk based only on the first mode shape (ICC, 2006).

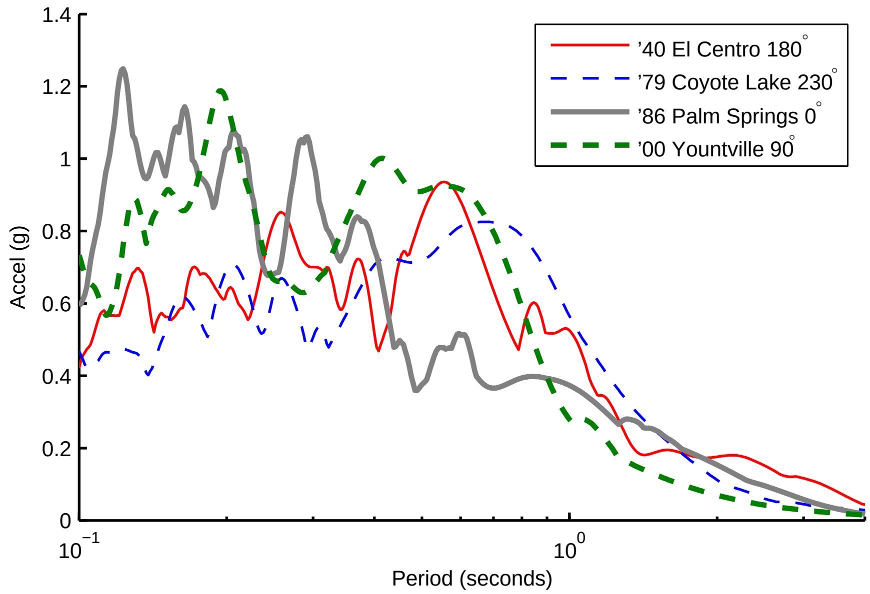

Figure 4: 5% Damped Elastic Response Spectrum (ERS) for Selected Acceleration Time Histories

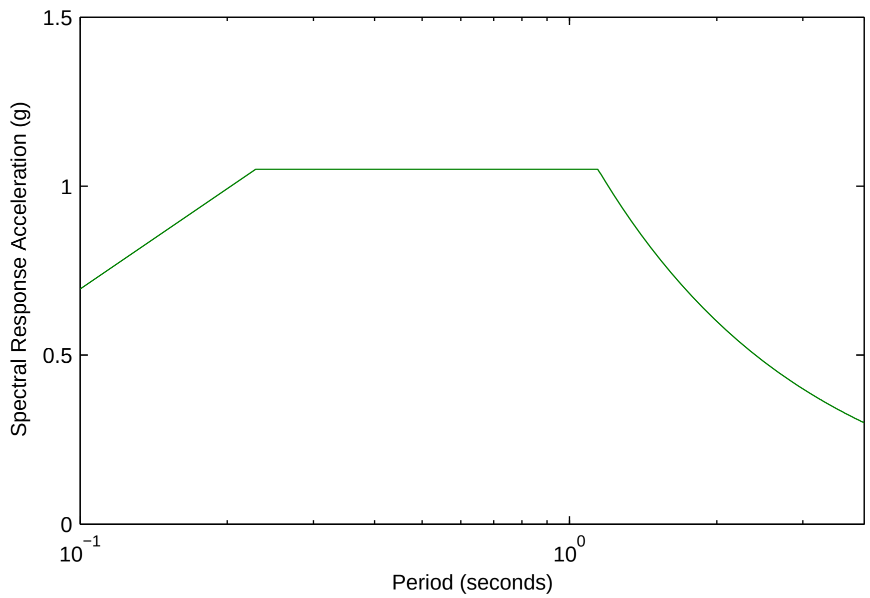

Figure 5: Example IBC 2006 Design Spectrum

Damping is a measure of a structure's ability to dissipate energy. A structure that has no damping would vibrate indefinitely while a structure that is critically damped would return to its un-deformed shape after a period of time equal to the first natural period. Most civil structures are under-damped which implies vibrations will oscillate about the structures un-deformed shape with a decrease in amplitude for each cycle. Friction and material dissipation are common sources of damping. Existing literature uses damping levels from 0.5% to 5% or more of critical damping for modeling wind turbines. This range is due to the dependence of observed damping for a turbine on wind speed and operating state among other factors.

Ductility is a measure of the amount non-linear deformation of a structure before the onset of collapse. Certification guidelines for wind turbines prescribe that seismic response is elastic (GL, 2003; IEC, 2005) thus implying that no non-linear deformation can occur. The primary intent of building codes is to protect human life by preventing collapse of civil structures. Design loads are lowered by building codes allowing the use of ductility, provided that sufficient deformation can be sustained without collapse. This practice results in more economical buildings while still protecting human life. For wind turbines there is little danger to human life in the event of collapse and the primary intent of design guidelines is to ensure reliable operation for the design life of the turbine. Non-linear deformation (damage) to the turbine without collapse interrupts reliable operation. The preclusion of damage removes ductility as a variable in design of a wind turbine. However, ductility is still important in the assessment of the probability of collapse for turbines.

3 Literature Survey

Seismic loading of wind turbines is being actively addressed in current literature. Guidelines for wind turbine design include earthquake considerations. Numerous models have been published to investigate seismic loading of turbines. Results from specific investigations have been published providing preliminary conclusions regarding seismic loading on turbines.

3.1 Existing Codes and Standards

Three main standards or guidelines provide direct guidance for seismic loading of wind turbines: Guidelines for Design of Wind Turbines (Risø, 2001); Guideline for the Certification of Wind Turbines (GL, 2003); and IEC 61400-1 Ed.3: Wind turbines - Part 1: Design requirements (IEC, 2005). Outside of these building codes, such as the 2006 IBC (ICC, 2006), can be used to inform earthquake analysis of wind turbines, but do not directly address wind turbines. All three turbine guidelines suggest that there are few regions throughout the world where seismic loads may be design driving. In all cases seismic analysis is only required in regions of high seismic hazard or as required by local authorities.

Of the three guidelines mentioned, Guidelines for Design of Wind Turbines (Risø, 2001) provides the most general suggestions. The Risø guidelines function as a basic introduction to the concerns associated with seismic loading. This is consistent with the general intent of the publication to serve as a detailed introduction to all engineering wind turbine subjects. A model for seismic analysis is proposed that accounts for the nacelle, rotor, and ¼ of the tower mass with a lumped mass at the top of the tower. It is suggested that the resulting period be used to select a spectral response acceleration from a design response spectrum that will be used to determine the seismic loads on the tower. This is a SDOF frequency domain analysis because of the simple model used (without any time domain simulations). The procedure is similar to simplified procedures used in building codes. No recommendation is provided for the appropriate level of damping. An assumed level of damping will be embedded in the design response spectrum used in analysis, which is typically 5% (ICC, 2006). No guidance is provided in translating the resulting spectral response acceleration into design loads. In the absence of specific guidance, it is assumed that appropriate building code procedures will be employed.

In contrast to the Risø guidelines, the Germanischer Lloyd (GL) guidelines (GL, 2003) provide little introduction to the specific details of earthquake engineering. These guidelines are more prescriptive and provide detailed guidance on particular aspects of seismic risk, which is consistent with the intent of the publication as a set of requirements for certification. The guidelines first suggest that either local building codes should be applied or the American Petroleum Institute (API, 2000) recommendations are to be applied in consultation with GL Wind. The guidelines then prescribe details of the required seismic analysis. A return period of 475 years is prescribed as the design level earthquake. The resulting earthquake load is to be combined with all normal external conditions with a safety factor of 1.0 for the earthquake load. Consideration of at least 3 modes is required for both time domain and frequency domain analyses. For time domain analyses at least 6 simulations must be performed per load case. Finally, it is prescribed that the tower is to behave elastically unless the tower has characteristics that allow ductile response, such as a lattice tower. As with the Risø guidelines (Risø, 2001), no guidance is provided regarding the level of viscous damping.

In a similar fashion to the GL guidelines (GL, 2003) the IEC guidelines (IEC, 2005) focus on prescribing requirements for analysis of seismic loads. The design level earthquake is prescribed as a 475 year return period event and the resulting loads must be superimposed with the greater of the lifetime averaged operating loads or the emergency shutdown loads. Consistent with building code (ICC, 2006) the GL guidelines required safety factor for the earthquake load is 1.0 (GL, 2003). The analysis may be conducted through time domain or frequency domain methods, but either method must use consecutive modes with a total modal mass of 85% of the total mass of the turbine. A simplified procedure is provided in Annex C, which is intended to be a conservative estimate of seismic loads. The procedure suggests the use of a design response spectrum from local building code adjusted to a damping of 1% to establish the design response acceleration based on the first tower mode. The force required to accelerate a mass equal to the combination of the rotor, nacelle, and half of the tower mass by the design response acceleration is applied at the tower top to determine a design base shear and moment. This value is then combined with the loads calculated for an emergency shutdown at the rated wind speed.

3.2 Turbine Modeling Methods

Existing literature regarding modeling wind turbines for seismic loading is divided between two types of models; models that focus on the tower by accounting for the mass of the nacelle and rotor as a point mass at the top of the tower; and models that describe the full turbine including the nacelle and rotor with some level of detail. Simplified models are attractive as they remove the complexity of modeling the rotor. The simplified approach casts the turbine as a SDOF system and may be unreliable for modeling behavior that arises from modes other than the first tower mode.

In contrast full system models increase complexity of interpreting results. The additional overhead is rewarded by the model flexibility. Existing full system models attempt to incorporate all possible factors to seismic risk including aerodynamic loads, rotor dynamics, soil-structure interaction, electrical system dynamics, and other sources. A full system model further has the benefit of prediction of component loads instead of only tower loads. Since there is no systematically documented experience of seismically induced failure in wind turbines, designers cannot be certain how a turbine might fail in a seismic event. It is generally assumed that the tower and foundation are the critical components for seismic loading. Full systems models can help evaluate component loads not included in a simple tower based model.

3.2.1 Simple Models

As previously discussed, both Risø National Laboratory (Risø, 2001) and the IEC Annex C (IEC, 2005) provide simplified procedures for estimating seismic loading of a wind turbine. The difference between these two procedures is subtle. The Risø procedure uses a simplified model to determine the first tower natural period that could prove useful for estimation during design iteration. The IEC assumes that the first natural period is known based on existing analysis. Both procedures then use this first natural period to extract the design response acceleration from a design response spectrum. The Risø procedure then leaves the designer free to select an appropriate method to translate the design response acceleration into seismic loads where the IEC procedure prescribes that this acceleration be translated into a base shear and moment as described in section 3.1.

3.2.2 Full System Models

Proper aerodynamic analysis of wind turbines requires a full system model. This has lead to mature and widely used full system models in the wind energy industry. Two notable modeling tools specific to the wind industry are GH Bladed (Bossyani, 2000) which is produced by Garrad Hassan (GH) and FAST (Fatigue, Aerodynamics, Structures, and Turbulence) (Jonkman and Buhl, 2005) which is developed by the United States National Renewable Energy Laboratory (NREL). Both GH Bladed and FAST have been validated by Germanischer Lloyd for calculating operational loads associated with typical load cases. Other high quality models are also used in the wind industry, such as the FLEX5 code (Hansen et al., 2005), and several others.

GH added a seismic module to GH Bladed in response to demand for estimation of loading at seismically active sites (Bossyani, 2000). GH Bladed does not model the turbine using a finite element method due to computational complexity, but instead uses a limited-degree-of-freedom modal model. Modal calculations are conducted in the time domain for the major components of the turbine. The resulting nodal forces for each mode are then calculated. GH Bladed has two methods for simulating seismic loading. The first method is to use recorded acceleration time histories. The second method uses an iterative procedure to produce a synthetic acceleration time history with an elastic response spectrum that closely resembles a specified design response spectrum. To further increase flexibility the user is able to specify a foundation stiffness to account for soil and foundation influences on the structural response. The end result is a comprehensive package that is able to simulate seismic response of a turbine in the time domain with any specified level of damping in combination with other load sources. This approach allows the designer to explore numerous loading scenarios and obtain a detailed understanding of the resulting structural loads.

FAST (Jonkman and Buhl, 2005) uses a combined modal and multibody dynamics formulation to simulate a turbines dynamic behavior. The equations of motion are solved using standard multibody dynamics formulations for elements whose flexibility is determined by the summation of mode shapes provided by the user. Currently FAST does not directly provide a facility to simulate seismic loading. Instead a generic framework is provided that allows the user to provide a custom-developed loading routine to be imposed at the base of the turbine. With the appropriate additions FAST is capable of providing a full system model for seismic loading.

Another software package, FLEX5, may be used to model seismic loading of wind turbines (Ritschel et al., 2003). FLEX5 was developed by Stig Øye from the Department of Fluid Mechanics at the Technical University of Denmark (Larwood and Zuteck, 2006). The implementation of FLEX5 is based on a modal formulation with selected degrees of freedom (Hansen et al., 2005). Simulations are conducted in the time domain and produce records of component loads and deflections.

Efforts have been made to use a hybrid multibody system (MBS) (Zhao and Maisser, 2006; Zhao et al., 2007) to develop a full system model for wind turbines. This approach casts the turbine as a series of rigid bodies connected flexible elements. A set of analytical governing equations can be derived for the turbine using this approach and Lagrange's equations of the second kind. This approach, though more mathematically rigorous, does not require external calculation of component mode shapes.

3.3 Existing Results

The growth of wind power has lead to an interest in addressing seismic loading of wind turbines. Early publications (Bazeos et al., 2002; Lavassas et al., 2003) considering seismic loading of wind turbines focused on loading of the tower based on simplified models that lumped the nacelle and rotor as a point mass. Gradually interest shifted from these simple models to more refined models that also consider loads for turbine components other than the tower (Ritschel et al, 2003; Witcher, 2005; Haenler et al, 2006; Zhao and Maisser, 2006).

In 2002, one of the first attempts to quantify the dynamics of wind turbines due to seismic loading was published (Bazeos et al., 2002). This publication presented extensive finite element modeling of a prototype 450 kW turbine with a 38 meter tall steel tower designed for installation in Greece. The tower was modeled in detail using shell elements as well as by a simpler model that used beam/column elements. Both models addressed the rotor and nacelle by adding a point mass at the top of the tower and used a viscous damping of 0.5%. Time history analyses of the two models were conducted and compared. The results from the two models showed good agreement, but the more detailed model was required for buckling analysis. Soil structure interaction (SSI) was investigated using springs, dampers, and added mass. The main outcome of the SSI analysis was to show a significant decrease in the frequencies at which the second and third tower bending modes occurred due to base fixity. The analysis concluded that seismic loading did not produce design driving loads.

In 2003, a detailed finite element investigation was published of 1 MW turbine with a 44 m tall steel tower and 52 m rotor diameter designed for installation in Greece (Lavassas et al., 2003).

The seismic loading in this investigation was based on a multimode linear analysis which used a design response spectrum from Eurocode 3 for a site in seismic zone II with rocky soil. The authors concluded seismic stresses were 60 percent lower compared to those developed by extreme wind loads for this level of seismicity. Again the rotor and nacelle were simplified to a point mass at the top of the tower. The authors speculate that seismic design could become critical in regions with higher seismic hazard and less favorable soil conditions.

Windrad Engineering published an analysis that considers seismic loads for components other than the turbine tower (Ritschel et al, 2003). The publication first looks at the seismic loads produced by a modal analysis of a simple distributed mass cantilever beam model of a 2.5 MW Nordex N80 wind turbine with an 80 m rotor diameter and 60 m hub height. This modal approach produced seismic loads that closely matched the contemporary IEC approach (IEC, 1999) based on a synthetic input time history with a PGA of 0.3 g. Realizing that such a model was incapable of properly addressing component loads, a full system model with 28 degrees of freedom was developed using FLEX5 by mapping ground acceleration through a coordinate transformation into effective external nodal forces. The FLEX5 model produced lower moment demand at the base of the tower compared to both the IEC approach and the modal approach using the same synthetic earthquake time history. The loads approached parity toward the top of the tower. At the top of the tower the seismic load from FLEX5 slightly exceeded both the IEC approach and the modal approach. The difference was not significant and it was concluded that the existing design loads were sufficient. The vertical seismic excitation caused higher bearing loads than those from extreme wind conditions. Vertical excitation was also found to induce tilt vibration in the nacelle. This investigation concluded that seismic loads in the blades were about 70% lower than those caused by the 50-year wind loads.

Witcher (2005) presents an overview of the GH Bladed seismic module in conjunction with some preliminary results for loading of a 2MW upwind machine with an 80 m diameter rotor and a 60 m tower. The results show the response in three load cases: continuous operation throughout the earthquake; emergency shutdown initiated during the earthquake; and parked throughout the earthquake. The difference in the resulting maximum moment demand at the base of the tower for the three load cases was compared for a time and frequency domain calculation. Only a 2.9% increase in the maximum moment demand was observed in the time domain compared to the demand from the frequency domain. A fourth load case considered the turbine parked while subjected to an earthquake in combination with high winds. This case resulted in a 79% increase in moment demand for a time domain simulation from that calculated using a modal simulation. This result is used to highlight the importance of time domain simulations to account for aeroelastic interaction.

Windrad Engineering presented initial results of their new software, Simulation of Wind Energy Converters (SIWEC), for simulation of wind turbine dynamics including seismic loading at the 2006 European Wind Energy Conference (Haenler et al, 2006). This paper shows how SIWEC will be capable of addressing wind industry requirements (IEC, 2005) as well as requirements introduced from Eurocode 8. Results are shown for simulation that subjects a turbine with an 80 m rotor diameter and 60 m hub height operating in 13 m/s wind to an earthquake with a 0.3 g PGA. The simulation also includes the dynamics associated with shutdown of the turbine. The full system model predicts modes at frequencies in the region of maximum spectral response acceleration for typical design response spectra. The results focus on the relative increase in higher mode response. It is noted that for normal wind loading 80% of the tower energy is associated with the first mode. During the earthquake simulation the energy in the first mode is reduced to only 54% percent of the tower energy. The analysis concludes that higher tower modes are more important for earthquake loading than typical wind loading.

In collaboration with Peter Maisser and Jingyan Wu, Xueyong Zhao presents what was termed a hybrid MBS for modeling turbine dynamics (Zhao et al., 2007; Zhao and Maisser, 2006). The technique is detailed in the 2007 publication (Zhao et al., 2007) by providing the theoretical development, showing resulting mode shapes, and variation in natural frequencies as a function of rotor speed for a 600 kW turbine with a 43 m diameter rotor and a 52 m tower. This introduction is followed by an extension of the technique to include seismic loading and soil structure interaction (Zhao and Maisser, 2006). The rotor, which was initially modeled with three flexible blades, was simplified to a rigid disk when considering seismic loading. The soil structure interaction was addressed by connecting the turbine base to a rigid support with translational and rotational springs and dampers whose properties were derived based on assumed soil properties. The response of a 1.5 MW turbine subjected to turbulent wind with a mean velocity of 10.16 m/s and an earthquake acceleration time history with a maximum acceleration of 0.06 g is calculated using this model. The low PGA is consistent with a minor or very distant earthquake. Negligible impact was observed for the tower base shear and bending moment. In contrast, oscillation in the lateral reaction force of the main bearing was significantly increased. This observation is similar to that of Ritschel et al. (2003) regarding vertical loads in the main bearing.

Experimental and numerical investigations of damping due to aerodynamic effects show a wide range of damping for turbines (Riziotis et al., 2004; Hansen et al., 2006). Theoretical predictions show that some turbines can exhibit negative damping depending on the wind speed (Riziotis et al., 2004). It appears that newer pitch controlled machines exhibit higher damping in the tower at the first natural frequency than older stall regulated machines (Riziotis et al, 2004). Existing publications show that in all cases aeroelastic effects lead to directional damping that is higher for longitudinal vibration than for lateral vibration. This predicted and observed directivity in damping suggests that seismic loading should be considered in both directions to ascertain if the decreased damping in the longitudinal direction will increase the resulting design loads despite not being directly additive with other loads.

3.4 Perceived and Actual Risk

Few strong earthquakes have occurred in the vicinity of utility scale wind farms. Two events of interest for seismic loading of wind turbines are the 1986 North Palm Springs Earthquake and 1992 Northridge Earthquake. Ground motion recordings from the vicinity of wind farms are available for both earthquakes. The North Palm Springs Earthquake occurred very near wind turbine installations situated to the North-West of Palm Springs. California Strong Motion Instrumentation Program (CSMIP) station 12149 was approximately 5 km from these wind farms. The 1992 Northridge earthquake occurred about 80 km from wind farms located in Tehachapi, California and was recorded by the CSMIP station 34237. This station is within 1 km of Tehachapi wind farms.

The Northridge Earthquake had a moment magnitude of 6.7 whereas the North Palm Springs Earthquake was weaker with a moment magnitude of 6.2. Monetary damage from for the Northridge Earthquake was greater, mainly due the proximity of the epicenter to a densely populated area. This supports the perception that the Northridge earthquake presented a higher risk to wind turbines than the North Palm Springs earthquake. The recorded earthquake records do not support this conclusion. The recording near the Tehachapi wind farms (Northridge) shows a peak ground acceleration of 0.06 g, which is similar to that investigated by Zhao and Maisser (2006) that showed minimal impact on resulting loads. Verbal reports indicate no damage to turbines in the area in agreement with published findings. In contrast the PGA recorded near the Palm Springs wind farms was 0.33 g, which represents a much more significant event in the range shown by numerical investigation to produce loads near design loads (Ritschel et al, 2003). This level of ground acceleration represents a much higher chance of damage to civil structures, including the wind turbines that were in the area. News reports from the 1986 North Palm Springs Earthquake do not detail any wind turbine damage, but document significant damage to buildings in the vicinity. Proximity to the earthquake epicenter is a primary factor for PGA experienced at a site and should be considered in site specific assessments of seismic loads for wind turbines.

3.5 Current Practice

A picture of the state of practice early in the decade for site specific seismic analysis of turbines was presented at the 2002 annual convention of the Structural Engineering Association of California (Agbayani, 2002). The publication details seismic loading based on the 1997 Universal Building Code (ICBO, 1997). Design response spectra are scaled from 5% viscous damping to 2% using FEMA-273 (FEMA, 1997). The article notes that in contrast to past turbines some newer units with increased size and weight are governed by seismic loads instead of wind loads in regions of high seismic hazard. This appears to be in conflict with finding presented later in this report. The source of this discrepancy is unclear.

4 Scaling Study

For this paper we use the simplified IEC method to estimate the dependence of tower moment demand on turbine size to provide a preliminary understanding of seismic risk (IEC, 2005). Four turbine models of varying size are modeled in FAST to determine their first period (Jonkman and Buhl, 2005). Table 1 summarizes the units. The larger 5 MW unit (Jonkman et al, 2009) is a hypothetical unit developed by the NREL to support concept studies of new turbines. The integrated European Union UpWind research program and the International Energy Agency (IEA) Wind Annex XXIII Subtask 2 Offshore Code Comparison Collaboration have adopted this 5 MW reference model. Period estimates are based on component mass and stiffness with no consideration of aerodynamics. Units documented in the literature presented above were included to broaden the scope of turbines considered. A summary of these units is shown in Table 2. When available information was insufficient the unit was omitted from plots and trend analysis.

Unit Type	*Rated Power (kW)*	*First Period (Sec)*
AOC-15/50	50	0.64
AWT-27 CR2	175	1.14
WP 1.5 MW	1500	2.47
NREL 5 MW Baseline	5000	3.17

Table 1: Basic Properties of Turbines Studied Using FAST

Unit Type	*Source*	*Rated Power (kW)*	*First Period (Sec)*
Prototype	Bazeos et al., 2002	450	1.04
Unknown	Zhao et al., 2007	600	1.48
Prototype	Lavassas el at., 2003	1000	1.67
Unknown	Zhao and Maisser, 2006	1500	2.28
Unknown	Witcher, 2005	2000	1.62 (estimated)
Nordex N80	Ritschel et al., 2003	2500	1.91

Table 2: Basic Properties of Turbines from Literature

The amount of energy a turbine can capture is directly proportional to the swept area of the rotor. This implies that the rated power is proportional to the rotor diameter squared or equivalently the rotor diameter is proportional to the square root of rated power. Figure 6 graphically shows the relationship between rated power and rotor diameter. In the determination of tower height there are many variables, but rotor diameter is a large contributor that should have approximately a linear relationship to the tower height. Again this is equivalent to the tower height being proportional to the square root of the rated power. The relationship between rated power and tower height is shown in Figure 7. The red fit lines show a best fit curve that is proportional to the square root of the rated power. There is considerably less scatter in the relationship between rated power and rotor diameter as expected. Because the tower height is also influenced by the local wind conditions the same turbine may use a different tower to accommodate site specific conditions.

Figure 8 shows the rated power versus the first period of the turbines. The red fit line assumes a power relationship between the rated turbine power and the first period. The IBC 2006 and other building codes use simplified equations to determine the first period of a structure depending on the type of structure along with the height of the structure. ASCE 7-05 (ASCE, 2006), the source referenced by IBC 2006 to estimate period, assumes a power relation between a structures height and the first natural period of the form:

$$T_a = C_t h_n^x$$

Where T_a is the resulting estimate of the first natural period based on the nominal height h_n. The constants C_t and x are dependant on the type of structure. Fitting the data for the turbines studied here to this formula yields a value of 0.015 and 1.183 for C_t and x, respectively when the tower height in meters is used as the nominal height (Figure 9). These values for C_t and x represent a best fit of the data whereas code values are intended to provide a conservative (lower bound) period estimate. A value of 0.022 and 1.05 for C_t and x, respectively provides a reasonable lower bound for the data investigated (Figure 9).

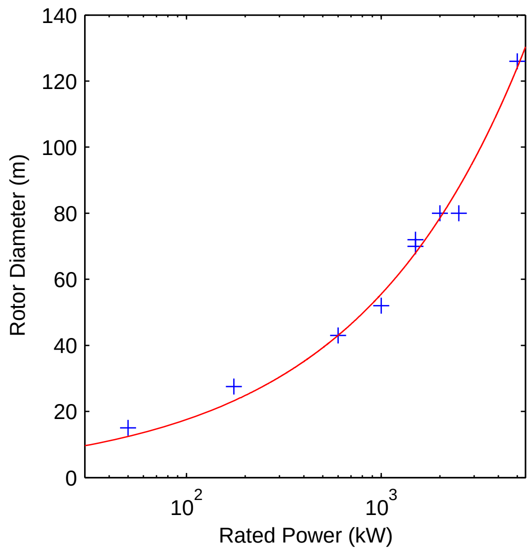

Figure 6: Rated Power vs. Rotor Diameter

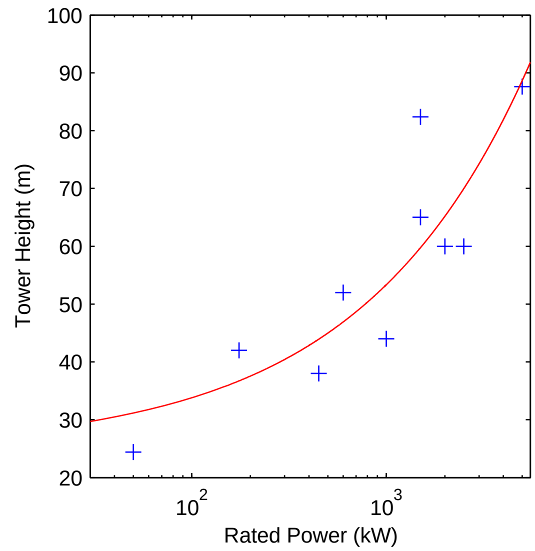

Figure 7: Rated Power vs. Tower Height

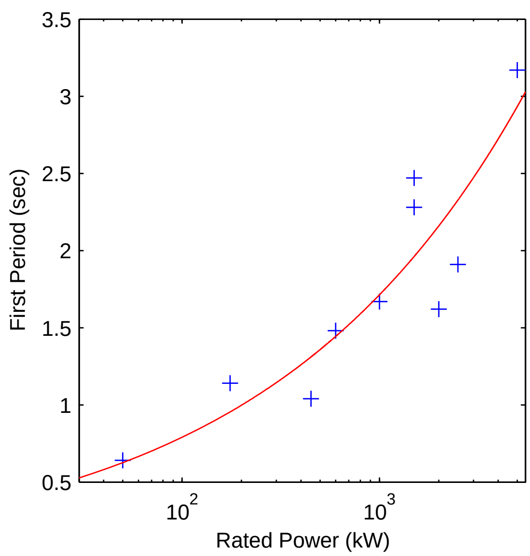

Figure 8: Rated Power vs. First Period

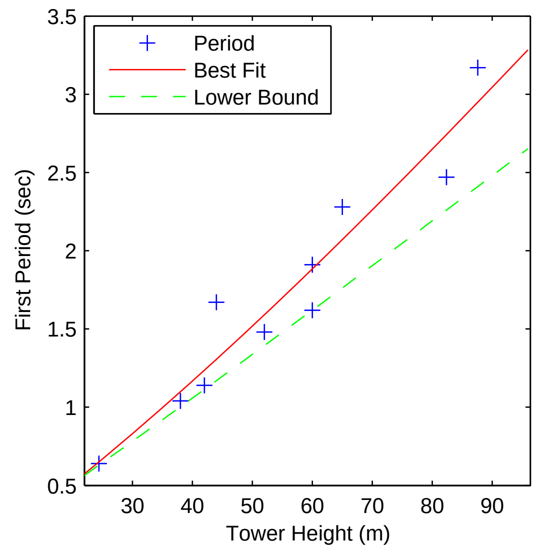

Figure 9: Tower Height vs. First Period

4.1 Seismic Demand from IBC 2006 Design Response Spectrum

Modern building codes allow the use of a design spectrum based approach to determine seismic loads for civil structures. This casts the structure as a SDOF system so that based on the calculated period and damping the resulting design response acceleration can be calculated. This acceleration is used to derive seismic base shear and moment demand.

This section presents a SDOF assessment using the procedure described in Annex C of the IEC design requirements for wind turbines in conjunction with the IBC 2006. Two locations in seismically active regions of California were investigated for comparison. The locations were Tehachapi and Palm Springs. Both locations currently have active utility scale wind farms.

4.1.1 5% Damped Design Response Spectrum

The Tehachapi site was a hypothetical location near existing wind farms in Tehachapi, California. The site was classified as a rock site based on IBC 2006 with a soil shear wave velocity between 760 and 1,500 meters per second. This resulted in the design response spectrum for a rock site is shown in Figure 10 based on seismic hazard maps from IBC 2006. These site characteristics lead to a moderate seismic hazard that may result in seismic loads controlling design for some structures.

The Palm Springs site was based on the published site conditions at the California Strong Motion Instrumentation (CSMIP) station 12149 located at the new fire station in Desert Hot Springs, California. A measured soil shear wave velocity of 105 meters per second results in a site classification as a class E site. The resulting design response spectrum based on IBC 2006 seismic hazard maps and soft soil conditions is shown in Figure 10. The site characteristics for the Palm Springs location lead to a high seismic hazard.

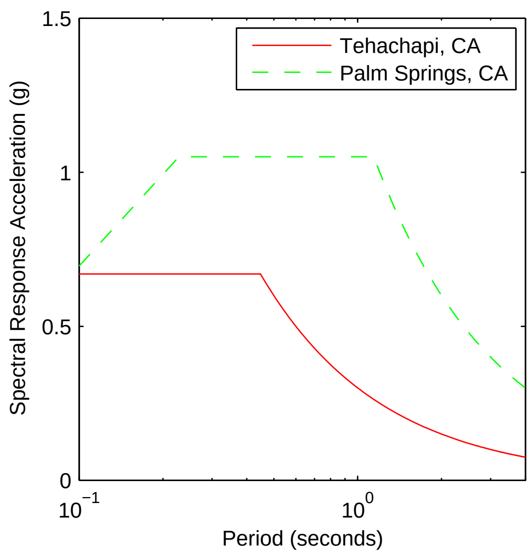

Figure 10: IBC 2006 Design Response Spectrum 5% Damped

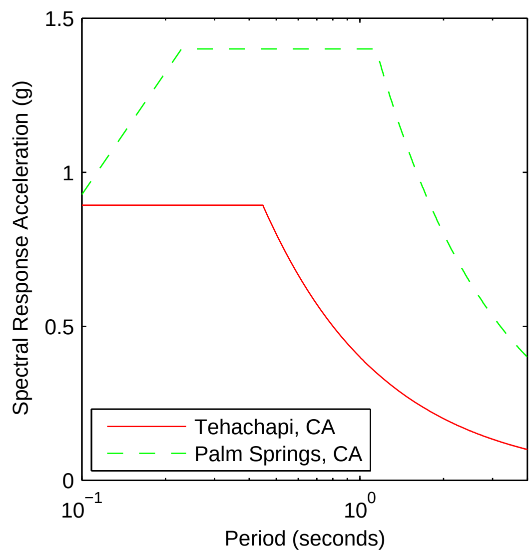

Figure 11: IBC 2006 Design Response Spectrum Adjusted to 1% Damped

4.1.2 Scaling Design Response Spectra From 5% to 1% Damped

As described before the IBC design response spectrum assumes a viscous damping level of 5% of critical (ICC, 2006). Currently discussion exists regarding the appropriate level of damping for use in modeling a turbine for seismic loading. Factors such as operational state and direction of base excitation in comparison to wind direction will directly influence the effective damping. Since this simplified assessment could not model these factors, 1% of critical damping, as suggested in IEC Annex C (IEC, 2005), was used. The 5% damped design response spectrum must be adjusted to account for the assumed 1% damping in the turbine. Most published methods (Newmark and Hall, 1982; Naeim and Kircher, 2001; FEMA, 1997) for scaling a design response spectrum use a damping adjustment factor, B, to scale the 5% damped design response spectrum, R_5, to the a design response spectrum with the desired level of damping, R_x, following the relation:

$$R_x = \frac{R_5}{B}$$

Often *B* is considered to be a function of structural period. Naeim and Kircher (2001) suggest based on an investigation of over 1,000 recorded acceleration time histories that it is not necessary to vary the adjustment factor with the period. Adjustment factors for design response spectra to account for differences in damping are published from 2% to 20%. Since the IEC recommended damping level of 1% is outside of this range, the adjustment factor was extrapolated to 1% which resulted in a value of approximately 0.75. Figure 11 shows the design spectra for Tehachapi and Palm Springs after being adjusted to a damping level of 1%.

4.1.3 Seismic Demand Based on Design Response Spectra

With the design response spectra adjusted to 1% of critical damping the design acceleration is selected based on the structural period. Figure 12 shows the resulting design acceleration as a function of rated power for the two sites considered. The horizontal portion of the fit presented for the Palm Springs site is a direct result of the plateau in the design response spectrum (Figure 11). The second component to assessing the seismic moment demand is the structural mass. Figure 13 shows both the head mass, which consists of the nacelle and rotor, as well as the point mass defined by IEC to calculate base moment. This point mass consists of the head mass and half of the tower mass (IEC, 2005).

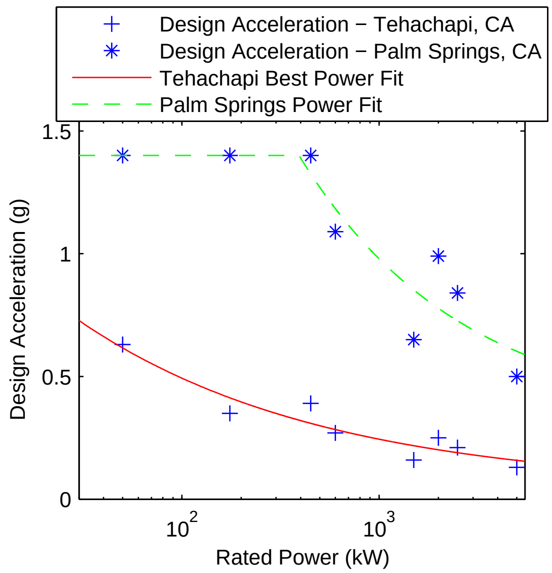

Figure 12: Rated Power vs. Design Acceleration

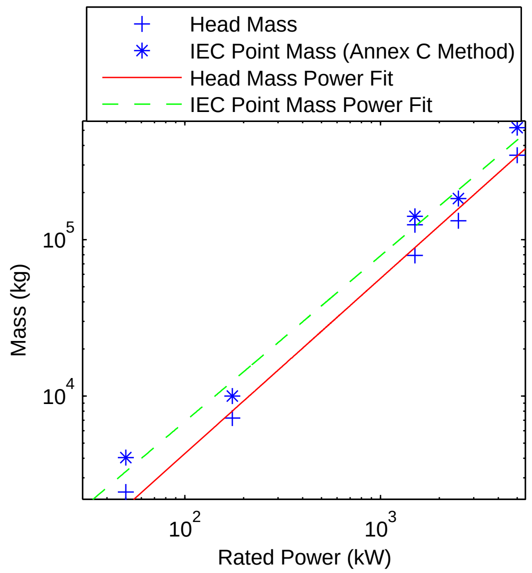

Figure 13: Rated Power vs. Mass

Finally the design acceleration is used with the point mass presented in Figure 13 to calculate a design base moment. The tower base moment is calculated as shown below.

Tower Base Moment = (IEC Point Mass)(Design Acceleration)(Tower Height)

Figure 14 shows the resulting design moment as a function of rated power for the two sites considered. The resulting moment from seismic demand appears to be growing proportionally to the rated power to the 1.1 power. If a turbine is scaled proportionally to increase the rated power the moment demand from wind loading scales with the swept area times the height. Using this proportional scaling and the previously assumed linear relation between rotor diameter and tower height the wind moment scales proportionally to the rotor diameter cubed. This scaling is equivalent to the wind moment scaling as the rated power to the 3/2 power. Figure 14 shows a band with moment growing as a function of the rotor diameter cubed to compare the anticipated growth of the moment demand from wind loading to the growth of the moment demand from seismic loading. The moment due to seismic loading using IBC 2006 and Annex C of the IEC design requirements (IEC, 2005) does not increase as rapidly as the anticipated wind moment. It should be noted that according to the IEC procedure the seismic tower base moment demand must be combined with the demand calculated for an emergency shutdown at the rated wind speed. This combination may still result in design driving loads even though the seismic moment alone is growing slower than the expected growth rate of the wind moment.

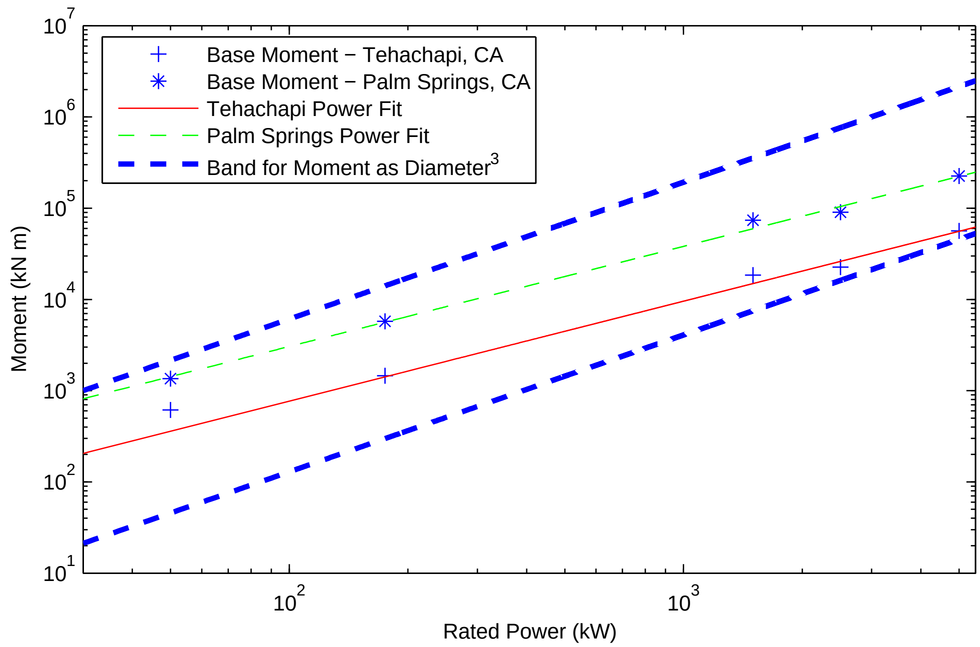

Figure 14: Rated Power vs. Bending Moment for 1% Damping

5 Conclusion

This paper presented the current situation for consideration of seismic loads for wind turbines. This subject is attracting more interest as the use of wind power grows, particularly in seismic regions. First an introduction to seismic risk was presented. A review of the existing literature on the topic provides an overview of research conducted to address seismic loads for turbines. Finally a preliminary analysis based on existing guidelines was conducted to understand how tower moment demand scales with rated power.

The literature shows a development from simple models that focus on predicting tower loads to full system models that illuminate loads for other components. The published analyses show that seismic loading may impact more than just the tower and suggest that full system models are important in analyzing seismic demand for turbines. Soil structure interaction has been incorporated into these full system models and exhibits a strong influence on higher modes. Of the work on seismic loading of turbines presented here, Haenler et al. (2006) is the only publication to mention validation with measured data. Testing, both nondestructive and destructive, and documented seismically induced damage is needed to provide valuable data to validate predictions and will serve as a basis to refine modeling techniques. In addition to reducing uncertainty in predicting seismic loads for wind turbines results will illuminate if, as predicted by presented research, higher tower modes are more important for earthquake loading than typical wind loading.

Dimensional analysis indicates that wind-driven loads grow faster than seismic-driven loads as machines get larger, as shown in Figure 14. However, engineers work diligently to keep growth of wind loads below the anticipated cube of the diameter. Smarter control systems, new materials, and other advances are regularly employed to achieve this goal. This effort may change the relationship between aerodynamic and seismic loads in future large machines. Thus, careful attention must continue to be paid to seismic loads in regions with high seismic hazard. Designs must also consider the seismic load as part of required load combinations, which may result in a design driving combination.

Good progress has been made in the systematic consideration of seismic and other load sources for wind turbines. With the availability of advanced aeroelastic modeling tools more detailed and accurate site specific analysis is possible. Continued vigilance will ensure that seismic and other load sources are appropriately considered as the state-of-the-art develops for modeling wind turbines. Consistent systematic treatment of turbine design loads will enhance the reliability and economic viability of wind turbines.

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