Seismic Analysis of Wind Turbine System Including Soil-structure Interaction

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ABSTRACT:

A numerical method is presented for the computation of artificial earthquake records consistent with any arbitrarily specified target response spectra (TRS) requirements. The proposed algorithm does not create new time histories but rather modifies, on the basis of an iterative deterministic approach, existing records to fit specific design requirements. The efficiency of the algorithm and the accuracy of the fitting process are substantially improved on the basis of a predictor–corrector type approach. Then, the behavior and the load capacity of the 1 MW horizontal wind turbine steel tower have been studied with the aid of a refined finite element model. The structure is analyzed for seismic loads representing the effects of gravity and possible site-dependent seismic motions. To evaluate the effect of the soil-structure interaction (SSI) on the dynamic performance of wind turbine system, a comparative finite element model taking the foundation and surrounding soil into consideration is also created. Comparative studies have been performed on the results of the above analyses and some useful conclusions are drawn pertaining to the effectiveness and accuracy of the various models used in this work.

KEYWORDS: Response spectrum; Narrowband time history; Wind turbine; Steel tower; Seismic analysis

1. Introduction

For more economic benefit from wind energy, a long lifetime for a wind turbine system is certainly expected, thus it is essential for the wind turbine system to avoid failure and to stand up expected accidents. In general, the design loads of wind turbine structures are based more on the wind forces. When the wind turbine system is installed in active areas of earthquake, its dynamic response characteristics to seismic excitations must be investigated and considered in the structure design.

In the seismic analysis, a customary means for specifying ground motion in earthquake engineering, particularly for the design of nuclear power plants, has been the so called “seismic response spectra”. They can also be used to compute the maximum linear response of a structure to the corresponding earthquake motion. However, a rigorous detailed study on the dynamic behavior of a linear or nonlinear structure requires a time domain analysis along with the use of an accelerograms consistent with the design response spectra of the particular site under investigation. The problem of generating such artificial accelerograms has been studied by a number of authors using a variety of methods (Tsai, 1972; Scanlan et al, 1972; Levy et al, 1976; Kost et al, 1978; Preumont, 1989). In all these cases, the proposed procedure consists of the iterative modification of a “sample” acceleration record until it matches the specified response spectra. The selection of the most suitable “sample” depends on many factors one of which is the method that will be subsequently used to modify it. For a more detailed review of the various available methods for generation of artificial earthquake records, the interested reader is directed to Preumont (Preumont, 1989).

The soil-structure interaction (Stejska et al, 1996; Wolf, 1997)must be considered in seismic analysis, because the dynamic behavior of a wind turbine structure during earthquake is affected by in interactions among three coupled subsystems: the wind turbine structure, the foundation, and the geologic media around the foundation. This leads to a whole soil-foundation-wind turbine structure system.

2. Simulation of earthquake ground motions

2.1. Computation of response spectrum

The response spectrum is a plot of the maximum response of all possible single degree-of-freedom systems due to a specified load function. It can take the form of any quantity of interest, e.g. displacement, velocity, acceleration, etc., and is usually plotted versus the natural frequency or period range corresponding to the system under investigation. Thus, the digital computation of seismic response spectra requires the repeated numerical solution of a single degree-of-freedom mass-damper-spring system to base motion, usually acceleration. In this case, the equation of motion of the idealized single degree-of-freedom system can be cast upon substitution of the relative displacement $u(t)=v(t)-v_g(t)$, natural frequency $\omega_0=\sqrt{k/m}$, and damping ratio $\xi=c/2\sqrt{km}$, into the form

$$\ddot{u}+2\xi\omega_0\dot{u}+\omega_0^2u=-\ddot{v}_g(t) \tag{1}$$

the solution of Eq. (1) can be accomplished via the use of a variety of methods, such as Duhamel's integral or a forward step-by-step integration in time. Once the relative displacement $u(t)$, for a specific value ω_0, has been computed by either one of the above methods, the corresponding displacement $S_d(\omega_0)$, pseudo-velocity $S_v(\omega_0)$, and pseudo-acceleration $S_a(\omega_0)$ spectral values can be computed as

$$S_d(\omega_0)=\max|u(t)|,\quad S_v(\omega_0)=\omega_0S_d(\omega_0),\quad S_a(\omega)=\omega_0^2S_d(\omega_0) \tag{2}$$

2.2. Comparing a computed response spectrum to a TRS

A criterion for acceptance of an artificial time history requires that its computed response spectrum envelopes a specified TRS. Accordingly, the computed response spectrum of an artificial time history is considered to envelop the TRS when no more than five points fall below and not more than 10% below the TRS. For the purpose of closely fitting the computed spectrum to the TRS, the iterative fitting process will continue until the above criteria are met and, in addition, no point of the computed spectrum exceeds the TRS by more than 15%.

2.3. Adjusting a computed time history to fit a TRS

For the modification of a time history record to fit a TRS, a method proposed by Preumont (Preumont, 1989) is adopted in this work. According to this approach, such modifications of the time history record come as a result of adjustments made to its Fourier transform. The Fourier transform of a time history record can be conveniently computed numerically using Digital Fourier Transform in conjunction with the Fast Fourier Transform (FFT) algorithm.

For the purposes of this work, the original time history record and its corresponding response spectrum, computed as in the previous sections, serve only as a starting point of an iterative process that continuously adjusts the computed response spectrum $S_a(\omega)$ to fit any chosen TRS $S_T^a(\omega_0)$. However, since the computation of a time history record directly from a response spectrum is not a uniquely defined process, the Fourier transform of the time history record is adjusted, instead. Thus, given an acceleration time history and its Fourier transform $A(\omega_i)$, a modified $A^{new}(\omega_i)$ is computed for a sequence of frequencies ω_i, as follows

$$A^{new}(\omega_i)=A(\omega_i)\frac{S_a^T(\omega_i)}{S_a(\omega_i)} \tag{3}$$

Since the response spectrum is computed at only a relatively small number of frequencies ω_0, a linear interpolation of the response spectrum is used so that the modification introduced by Eq.(3) is performed over the much larger set of frequencies ω_i, required for the Fourier transform record $A(\omega_i)$ or $A^{new}(\omega_i)$. The above procedure assumes a linear relationship between the Fourier transform and the response spectrum adjustments. However, it is possible to speed up the convergence of the iterative process by assuming a nonlinear relationship. On the basis of the results obtained after two successive iterations, and $S_a^i(\omega_i)$ and $S_a^{i+1}(\omega_i)$, an improved correction factor can be computed assuming an exponential relationship between the Fourier transform and the response spectrum adjustments. This assumption yields the following formula

$$A^{new}(\omega_i) = A(\omega_i)\left[\frac{S_a^T(\omega_i)}{S_a(\omega_i)}\right]^q \tag{4}$$

where,

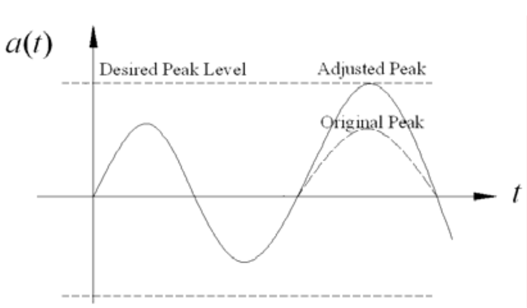
$$q = \frac{\log\left[\dfrac{S_a^T(\omega_i)}{S_a^{i+1}(\omega_i)}\right]}{\log\left[\dfrac{S_a^T(\omega_i)}{S_a^i(\omega_i)} \Big/ \dfrac{S_a^T(\omega_i)}{S_a^{i+1}(\omega_i)}\right]} \tag{5}$$

as $A^{new}(\omega_i)$ has been computed, inverse Fourier transformation is used to compute a newly modified acceleration time history.

In general, the peak acceleration of the modified time record will not be at the same level as the desired peak acceleration of the original time record. However, this characteristic peak value $\max a_g$ of the original accelerogram, which according to current design practices is site-specific, can be restored either by scaling or clipping the new record. In an effort to avoid extensive alteration of the frequency content of the computed accelerogram, the following scaling (Figure 1) and clipping (Figure 2) techniques are applied

1. If $\max|a_g(t)| \leq \max a_g$, all the values between the two zero crossings where the $\max|a_g(t)|$ occurs are scaled up by the ratio $\max a_g / \max|a_g(t)|$. This scaling procedure has been recommended by Preumont (Preumont, 1989).

2. If $\max|a_g(t)| > \max a_g$, the part of a_g outside the $\pm\max|a_g(t)|$ range is folded into the range. This technique has been introduced by Shinozuka (Shinozuka et al, 1988) and is known as "fractional folding".



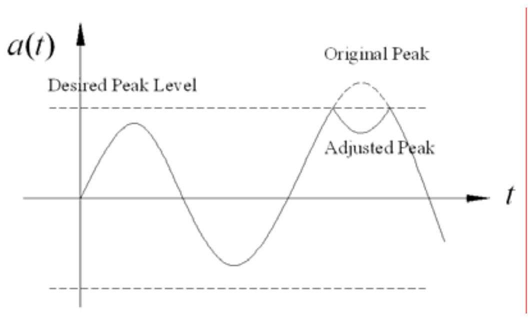


Figure 1 Partial scaling of computed acceleration Figure 2 Fractional folding of computed acceleration

Of course, the entire process can be repeated until the computed response spectrum fits the TRS to the satisfaction of the adopted acceptance criteria.

3. Geometry of the structure

The steel tower under investigation is the prototype of a group of steel wind turbine towers that is in the wind park at Mount Kalogerovouni in Laconia, Greece at an altitude of 800-1050m. In particular, a 1 MW capacity three-bladed cantilevered rotor is mounted on the top of the steel tower. The tubular tower has a total height of 44.075 m and is formed as a truncated cone with an external diameter of 3.30 m at the base and 2.10 m at the top. The shell thickness of the steel tower ranges from 18mm at the base to 10mm at the top.

4. Seismic analysis

4.1.Static loading

Two sets of static loads are considered: a) The pseudostatic aerodynamic loads under survival and operational conditions suggested by Riziotis (Riziotis et al, 1996). The concentrated aerodynamic loads at the elevation of the power transmission axis (el. +38 m), listed in Table 1, are due to the wind resistance and/or operation of the runner. In addition, aerodynamic loads distributed along the body of the tower itself have been computed by Riziotis (Riziotis et al, 1996) and are accounted for in the analysis under survival conditions. b) The second set of static loads due to gravity, consists of a concentrated load at the top of the tower representing the weight of the nacelle, runner, generator, gear box, etc. (236 kN with 0.75 m eccentricity along the *x*-axis) and the weight of the tower itself distributed along its height (78,500 N/m^3 specific weight of steel). The safety factors for the static loads are specified as (Germanischer Lloyd, 1993): Favorable gravity loads: 1.00; Unfavorable gravity loads: 1.35; Aerodynamic loads: 1.50.

Table 1 Pseudo-aerodynamic loads at elevation +38

	Fx(kN)	Fy (kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
Survival condition（70m/s）	216.23	0.28	0	96.03	82.92	5.38

Tall steel towers are usually designed considering the effect of wind loads as the only source of environmental dynamic disturbances. The effect of earthquakes as a possible source of damage or loss of serviceability is often neglected, even in high-risk seismic areas. However, neglecting earthquake effects should at least be justified by means of appropriate methods of analysis in conjunction with the recommendations of applicable building codes.

4.2.Seismic loading

The seismic loads are in accordance with the specifications of the EUROCODE8 (European Committee for Standardization, 2003). The corresponding elastic design spectrum of horizontal acceleration $S_d(T)$ is defined as

$$
\begin{aligned}
S_d(T) &= a_g S\left[\frac{2}{3}+\frac{T}{T_B}\left(\frac{2.5}{q}-\frac{2}{3}\right)\right], \quad 0 \le T \le T_B \\
S_d(T) &= a_g S\frac{2.5}{q}, \quad T_B \le T \le T_C \\
S_d(T) &= a_g S\frac{2.5}{q}\frac{T_C}{T}, \quad T_B \le T \le T_C \\
S_d(T) &= a_g S\frac{2.5}{q}\frac{T_C T_D}{T^2}, \quad T_C \le T \le T_D
\end{aligned} \tag{6}
$$

where a_g is the design ground acceleration on type A ground, a_g =0.35g; S is the soil factor, S =1; q is the behavior factor, q =1; T is period in seconds, T =1; T_B, T_C, T_D is characteristic cut-off periods for different soil conditions, T_B =0.15s, T_C =0.4s, T_D =2s.

In the study, artificial accelerograms compatible with the EUROCODE8 are developed with the method mentioned above (Figure 3, Figure 4). Subsequently, the seismic analysis of the tower is carried out in direct time domain. In addition, the influence of the soil structure interaction upon the dynamic characteristics of the tower, as compared to the fixed base analyses, is calculated in the following.

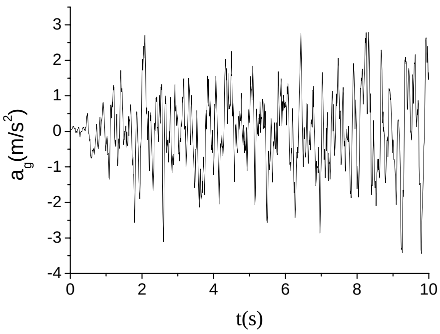


Figure 3 Artificial earthquake record

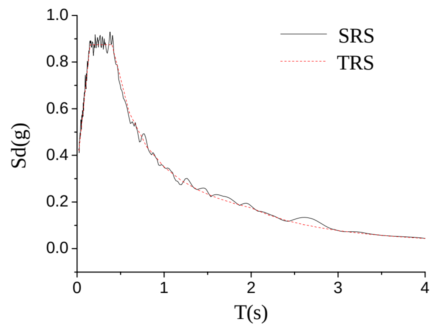


Figure 4 Computed and target response spectrum

4.3.Dynamic responses

The effect of the SSI on the dynamic characteristics of the structure under investigation can be assessed by introducing a set of discrete springs and dashpots at the soil–foundation interface, for example, Mulliken (Mulliken et al, 1998). In this case the previously discussed finite element models are modified by adding stiffening and damping elements at the foundation–soil interface and taking into consideration the mass of the foundation, as shown in Figure5. In addition, a portion of the soil mass, considered to move 'in phase' with the foundation should be taken into account (Mulliken et al, 1998).

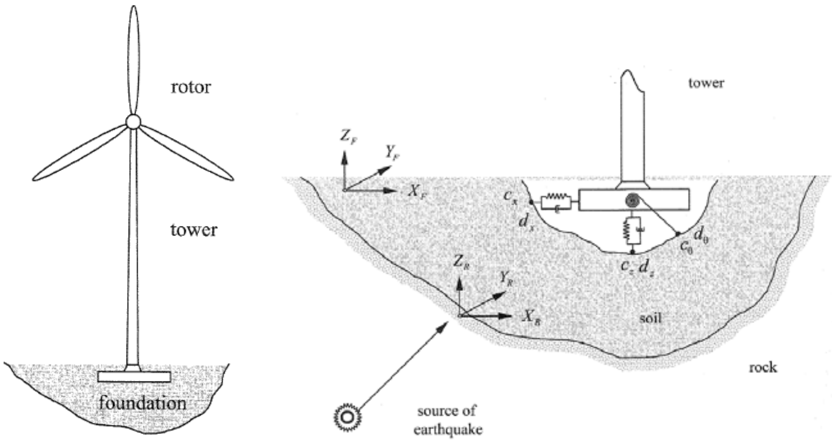


Figure 5 Model for seismic analysis including SSI

For the purposes of this study semi-rock soil conditions are assumed at the particular installation site with Poisson ratio 0.3, specific weight 21 kN/m^3 and shear modulus 520 MPa. A foundation proposed for the particular tower structure is a $10\times10\times1.80\text{m}^3$ concrete block. And the footing is assumed to be rigid. Based on the previous soil properties, foundation geometries and the related formulae proposed by Mulliken (Mulliken et al, 1998), the spring constants and added soil mass are computed as listed in Table 1.

Table 1 Spring constants and additional soil mass for the two most prominent modes of vibration

	Spring constant	Additional mass
Horizontal	14070×10^3 kN/m	177×10^3 kg
Vertical	17457×10^3 kN/m	593×10^3 kg

the seismic response of the tower structure to the design seismic motions described in the previous section of this work have been computed using: (a) the refined finite element model considering SSI effect and (b) the refined finite element model not considering SSI effect of in a direct time domain analysis. These two computations produce essentially different results, representative samples of which are shown in Figure 6. The domination of the seismic response of the structure by the first mode of vibration becomes obvious. According to these results, it can be found that

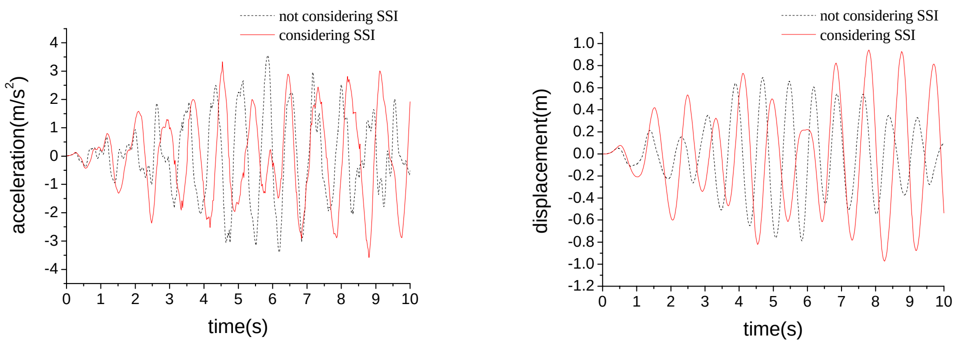


Figure 6 Influence of SSI on tower top acceleration and displacement

1) The influence of ground motions on wind turbine system can not be neglected. The maxima displacement at the tower top if one considers the SSI effect is 0.942 m, which approximates 2.14% of the tower height. So

neglecting possible earthquake action and assuming the wind loading as the only dominant loading is not persuasive enough.

2) SSI makes the displacement at the tower top change rapidly. The maxima displacement at the tower top if one considers the SSI effect is 0.942 m. However, the maximum displacement at the same position of the same model gives the result of 0.691m, which is only 73.4% of the one considering the SSI effect.

5. Conclusions

1) The "blades-tower-foundation" integrated model of wind turbine system is established. Two comparative FE models are adopted to investigate the SSI effect on the dynamic response of the system.

2) The influence of ground motions on wind turbine system is not negligible and should be put predominant position in the design of wind turbine system located at seismically active zone.

3) Based on the integrated model, the seismic analysis is carried out. According to the analysis results, the SSI effect significantly influences the global dynamic performance of the system and should never be neglected.

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