The 2013 World Congress on
Advances in Structural Engineering and Mechanics (ASEM13)
Jeju, Korea, September 8-12, 2013

Torsional-translational behavior of irregular wind energy structures

*Triantafyllos Makarios[1)] and Charalambos Baniotopoulos[2)]

[1)] *Earthquake Planning & Protection Organization, ITSAK, Thessaloniki, Greece*
[2)] *School of Engineering, University of Birmingham, UK and Department of Civil Engineering, Aristotle University of Thessaloniki (A.U.Th.), Greece*
[1)] *makariostr@ath.forthnet.gr*

ABSTRACT

In the present paper, the torsional-translational behavior of a prototype wind turbine tower considered as an irregular structure is examined. The last years a plethora of fatal failures of such structures occurred due to torsional dynamic actions. An appropriate model of the prototype irregular wind turbine tower by shell finite elements is herein developed, which has been verified by the application of the continuous model method for both the case of a fixed and the case of partially fixed foundation. As is well-known, the role of the higher eigenshapes is very important and may become critical in the case that the tower is subjected to strong dynamic loading, as is the wind loading, whereas it simultaneously is excited by a strong seismic motion. Thus, in order to estimate the role of fundamental torsional mode shapes of the above mentioned structure in the overall structural response of the tower, seven pairs of appropriately selected artificial seismic accelerograms that have response acceleration spectra (for ratio damping 0.05) equivalent to elastic acceleration spectra as proposed by Eurocode EN 1998-1 are used. To this end, applying a type of backwards analysis, an equivalent static torsion loading is defined.

1. INTRODUCTION

Wind energy structures are in the near future expected to correspond to the most significant part of energy produced by renewable energy systems (Baniotopoulos *et al*, 2010; Garch & Twele, 2012). To this end, an incredibly large number of Aeolian parks with numerous wind energy towers have been planned for erection. Such towers correspond to heights from 80m to 200m, most of them having a thin-walled steel tubular form with reduced thickness along their heights.

The abovementioned towers seem often to be rather simple for analysis. However, a lot of irregularities appears due to (a) the concentrated mass on the top, (b) the blades, (c) the dynamic loadings as are e.g. the wind-loading or the earthquake-loading and (d) the peculiarities of their foundation (Lavassas *et al*, 2003). In a tower analysis several significant design issues (e.g. the local buckling analysis of the shell structure, the

[1)] Senior Researcher of ITSAK and Lecturer of Dept. of Civil Engineering, A.U. Th.
[2)] Professor

stress concentration states around the openings and the tower modal analysis) that have to be thoroughly examined (Baniotopoulos *et al*, 2004).

Fig. 1 Torsional Failure of a Wind Energy Tower

Within this framework for the improvement of the analysis and design of the new generation wind energy towers, a modal analysis of a prototype tubular tower with fully fixed foundation was recently carried out by applying the continuous model method (Makarios & Baniotopoulos, 2013a), whereas the role of a partially fixed foundation was also examined by the same approach (Makarios & Baniotopoulos, 2013b). As a matter of fact, the complexity of the structural analysis issues at hand of then request sophisticated and innovative treatment (cf. e.g. Stavroulakis *et al*, 2006; Moutsopoulou *et al*, 2009). Recently, an advanced mathematical model of a horizontal axis wind turbine with flexible tower and blades was developed by Kessentini *et al* (2010), where the eigenvalue problem was treated both analytically and numerically by applying the differential quadrature method. It has to be also underlined that the use of the finite element method for the tower analysis in combination with the identification of tower dynamic characteristics via ambient vibrations is nowadays a standard technique for the substantiation of the tower modeling (Bayraktar *et al*, 2011). From a structural engineering point of view, a wind energy tubular tower can be idealised as a cantilever with cross-sections forming a thin-walled cylindrical shell. It is worthy noting that a plethora of fatalities due to dynamic torsional behavior of the wind energy towers have been the last decade occurred (Fig. 1). Appropriately developed models have been herein used in order to study the torsional behavior of such towers; using seven pairs of suitably chosen artificial seismic accelerograms that have response acceleration spectra (for ratio damping 0.05) equivalent to elastic acceleration spectra as proposed by Eurocode EN 1998-1 are used. This way, using a type of backwards analysis, an equivalent static torsion loading is calculated.

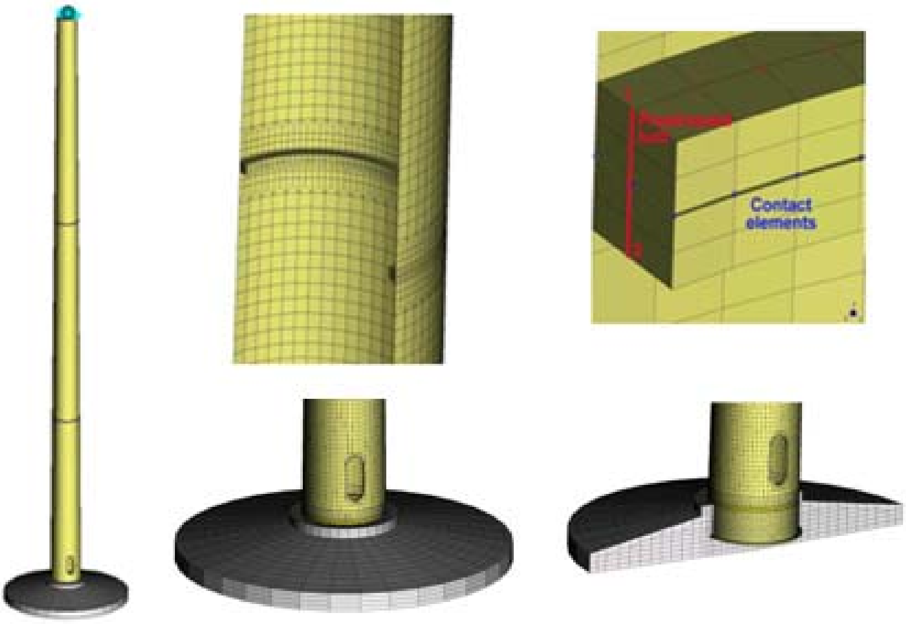

Fig. 2 Details of the wind energy tower

2. FORMULATION OF THE PROBLEM

2.1 Theoretical basis

In order to theoretically examine the torsional behavior of a steel tubular wind energy tower, the Technical Torsion Theory of a cylinder-cantilever with section radius *R* is considered loaded with a torsional moment M_t at its top(Beer & Johnston, 1992) (Fig. 3). Since each section of the cantilever is loaded with the same torsional moment M_t , the relative rotation of this cantilever is constant along all its length *L*. Consider an infinitesimal element *dz*, where the line AB becomes AB' while the shear deformation γ_R being the angle (the second order infinitesimals are ignored) given by the form

$$\gamma_R \approx \frac{BB'}{dz} = R \cdot \frac{d\varphi}{dz} \tag{1}$$

Thus, the shear stress τ_R yields:

$$\tau_R = G \cdot \gamma_R \tag{2}$$

where *G* is the shear modulus, namely $G = E/2(1+v)$ where *E* is the elasticity modulus and *v* is the Poisson index.

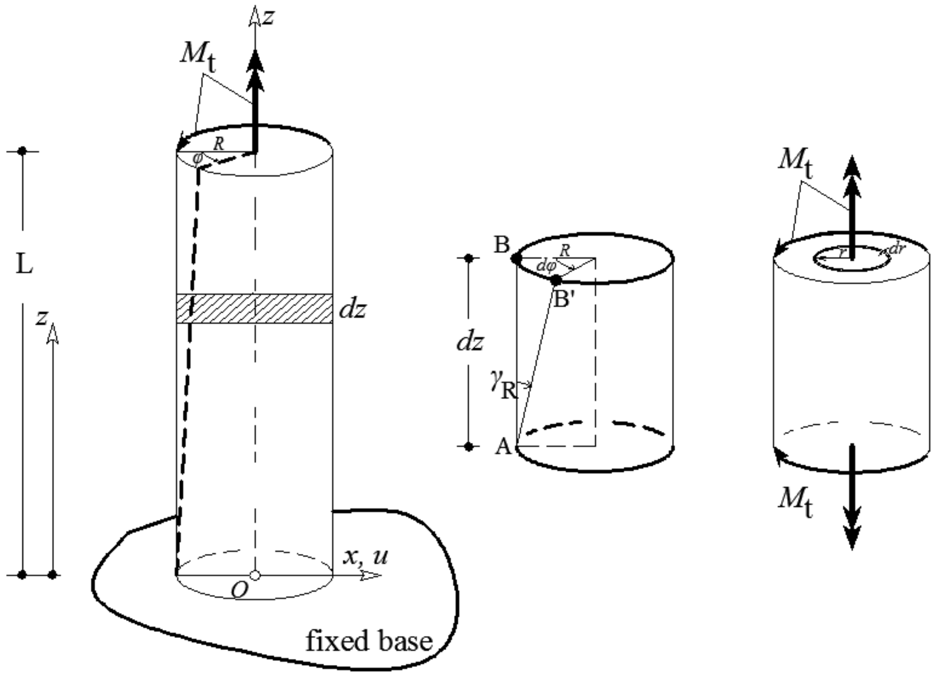

Fig. 3 Behavior of a regular cylinder loaded with a torsional moment M_{t} at the top

From Eqs(1 & 2) the following form is obtained

$$\tau_{\text{R}} = G \cdot R \cdot \frac{d\varphi}{dz} \tag{3}$$

In addition, the shear stress $\tau(r)$ along the radius *R* for $r \leq R$ is given as (Fig. 3):

$$\tau(r) = G \cdot r \cdot \frac{d\varphi}{dz} \tag{4}$$

By means of the equilibrium equations, the total moment of the internal shear stresses of a section $\tau(r)$ must be equal with the external torsional moment M_{t}. Thus,

$$\int_{r=0}^{r=R} \tau(r) \cdot [2\pi \cdot r \cdot dr] \cdot r = M_{\text{t}} \tag{5}$$

Inserting Eq.(4) into Eq.(5) the following form is obtained:

$$\int_{r=0}^{r=R} G \cdot r \cdot \frac{d\varphi}{dz} \cdot [2\pi \cdot r \cdot dr] \cdot r = M_{\text{t}} \tag{6}$$

Thus, Eq. (6) becomes:

$$2\pi \cdot G \cdot \frac{R^4}{4} \cdot \frac{d\varphi}{dz} = M_{\mathrm{t}} \quad \Rightarrow \quad \frac{d\varphi}{dz} = \frac{M_{\mathrm{t}}}{G \cdot I_{\mathrm{d}}} \tag{7}$$

where I_{d} is a geometric magnitude that is dependent on the section form which in the case of a circular section is equal with the polar moment of inertia I_{p} (namely, $I_{\mathrm{d}} = \pi \cdot \frac{R^4}{2} = I_{\mathrm{p}}$).

2.2 Equation of motion without damping of a thin-walled cantilever with reduced-section along its height

Consider the cantilever of Fig. 4 that has a thin-walled section reduced in elevation, loaded by a distributed dynamic torsional-loading $\mu_{\mathrm{t}}(z,t)$. The polar moment of inertia $\overline{I}_{\mathrm{p}}(z)$ and the mass moment of inertia $\overline{J}_{\mathrm{m}}(z)$ of a section are functions of the height *z*. Consider an infinitesimal element of the cantilever having length *dz*, where the internal torsional moments on the element are as depicted in Fig. 4. On this infinitesimal element, the inertia torsional moment $\overline{M}_{\mathrm{t},a} = \overline{J}_{\mathrm{m}}(z) \cdot \frac{\partial^2 \varphi(z,t)}{\partial t^2}$ is formulated by applying the D'Alembert's principle and $M_{\mathrm{t}}(z,t)$ is the torsional moment according to the well-known aforementioned Technical Torsion Theory. Equilibrium of vectors of torsional moments on the differential element in z-direction yields:

$$-M_{\mathrm{t}}(z,t) + \mu_{\mathrm{t}}(z,t) \cdot dz + [M_{\mathrm{t}}(z,t) + \frac{\partial M_{\mathrm{t}}(z,t)}{\partial z} \cdot dz] - \overline{J}_{\mathrm{m}}(z) \cdot \frac{\partial^2 \varphi(z,t)}{\partial t^2} \cdot dz = 0 \quad \Rightarrow \tag{8}$$

$$\frac{\partial M_{\mathrm{t}}(z,t)}{\partial z} + \mu_{\mathrm{t}}(z,t) - \overline{J}_{\mathrm{m}}(z) \cdot \frac{\partial^2 \varphi(z,t)}{\partial t^2} = 0 \tag{9}$$

Inserting Eq.(7) into Eq.(9), the form below is obtained:

$$G \cdot \frac{\partial [I_{\mathrm{d}}(z) \cdot \partial \varphi(z,t)]}{\partial z^2} + \mu_{\mathrm{t}}(z,t) - \frac{\overline{m}(z)}{A(z)} \cdot \overline{I}_{\mathrm{p}}(z) \cdot \frac{\partial^2 \varphi(z,t)}{\partial t^2} = 0 \tag{10}$$

where $\overline{J}_{\mathrm{m}}(z) = \frac{\overline{m}(z)}{A(z)} \cdot \overline{I}_{\mathrm{p}}(z)$ with $\overline{m}(z)$ is the mass per unit length in elevation and $A(z)$ is the area of the section in level *z*.

Eq. (10) is an utmost complex equation and in addition, a real wind energy tower does have more irregularities like as concentrated mass on the top in eccentric position and three diaphragms about vertical *z*-axis. All these do not permit the mathematical treatment of Eq. (10) and thus, a numerical approach by means of an appropriate model of shell finite elements seems to be a unique approach. As a matter of fact, in order to substantiate an effective model to simulate the structural response of a prototype wind energy tower, a continuous model method has been performed for both the fully fixed foundation (Makarios & Baniotopoulos, 2013a) and the partially fixed one (Makarios & Baniotopoulos, 2013b). This way, two effective numerical models has respectively resulted. In the present paper, having as aim to examine the torsional behavior of the prototype wind energy tower, the abovementioned two numerical models are used.

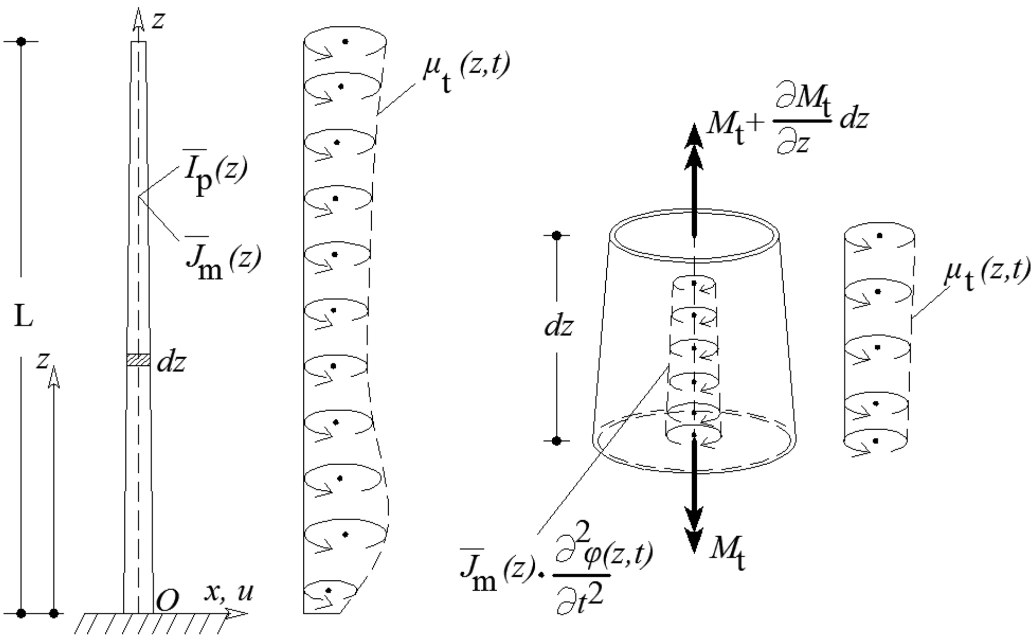

Fig. 4 Behavior of a regular tower loaded with torsional moments along its height

3. MODELLING USING SHELL FINITE ELEMENTS – MODAL ANALYSIS

3.1 Data of the wind energy tower

The prototype of the thin-walled wind tower at hand carries a 2 MW wind turbine. The height of the tower is *L*=80m and the total height of the wind turbine including the rotor and the blades is 125 m. The shell diameter at the base is 4.30 m linearly decreasing till the top where the tower diameter is 3.0 m. Shell thicknesses vary from 30 mm at the bottom to 12 mm at the top, linearly. The steel quality of the structure is S355, while its modulus of elasticity is *E*=210 GPa. Moreover, the self-weight of the tower is 1480 kN and the blade self-weight is $W_o = 1067.\ \mathrm{kN}$, located horizontally in a distance of 0.73 m from the vertical tower-axis passing from the tower cross-section centroid. Moreover, along the vertical direction, the weight of $W_o = 1067.\ \mathrm{kN}$ is located 0.50 m above the upper section of the tower.

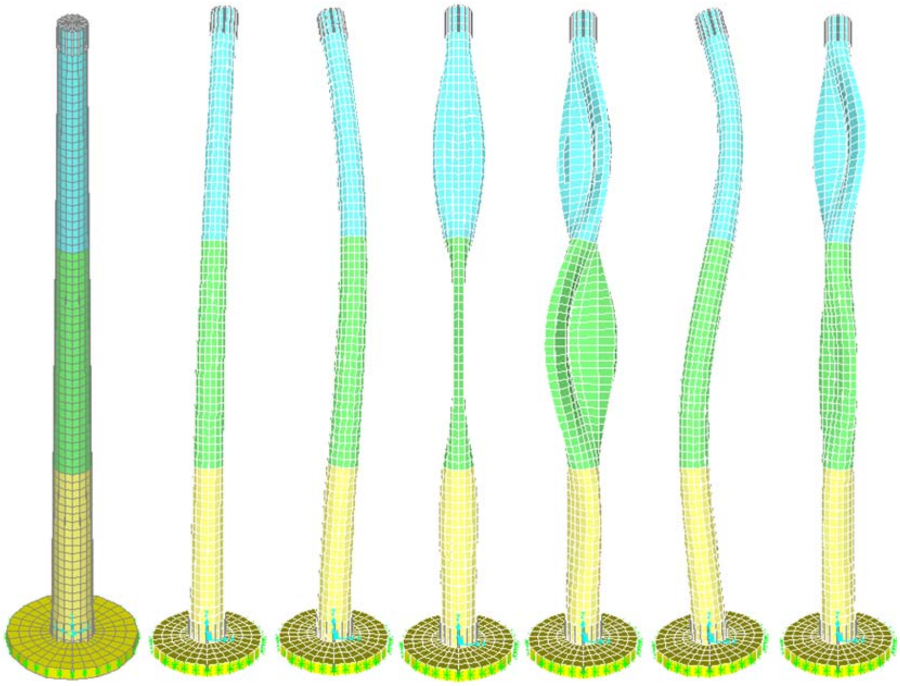

Fig. 5 Model of the prototype wind energy tower and the six first mode shapes with periods 2.59s, 0.33s, 0.135s, 0.131s, 0.12s and 0.09s respectively

3.2 Model& Modal Analysis

The model of the wind energy tower using shell finite elements along with the first mode shapes are also shown in Fig. 5. It is clear that the activation of the 3rd, 4th and 6th mode shape due to the wind loading and the rotation of the blades in neighbouring frequencies of the abovementioned critical mode shapes is the main cause of torsional failures as shown in Fig. 1. Thus, the critical frequencies of the modal analysis are

$$f_3 = \frac{1}{T_3} = \frac{1}{0.135} = 7.41\ \text{Hz}, \quad f_4 = \frac{1}{T_4} = \frac{1}{0.135}1 = 7.63\ \text{Hz}, \quad f_6 = \frac{1}{T_6} = \frac{1}{0.09} = 11.11\ \text{Hz}$$

Therefore, the critical ranges where the torsional or multi-torsion phenomena taking place are above the 5.00Hz as 7.41/1.50=4.94Hz, where 1.50 a safety factor that accounts for the alteration of frequencies due to possible erroneous assumptions and outcomes of the analysis. Moreover, the use of additional diaphragms in elevation for instance of diaphragms each 10.00m, contributes to a safer design strategy against torsional failures of the tower.

4. RESPONSE HISTORY ANALYSIS

In order to obtain the seismic target-displacements (peaks of translational and torsional displacements at the tower top) of the prototype wind energy tower for the design earthquake, the linear response history analysis has be performed using seven pairs of artificial accelerograms. Each pair constituted by two accelerograms being statistically independent (sect.3.2.2.1(3)P/ EN 1998-1). In order to form the seven pairs of accelerograms, five accelerograms (AS1, AS2, AS3, AS4 & AS5) have been used (Fig. 6). Each accelerogram is fully compatible according to requirements of sect. 3.2.3.1.2/EN1998-1(Fig. 7), whereas the considered soil category is considered of type *D* for the Hellenic seismic hazard zone I (*a*=0.16g) with total duration 25s. The correlation factor of each pair has been calculated: For pair AS1-AS4 r=0.007, for pair AS1-AS5 r=0.0093, for pair AS1-AS2 r=-0.0226, for pair AS3-AS4 r=0.0349, for pair AS4-AS5 r=0.041, for pair AS1-AS3 r=0.0577 and for pair AS2-AS4 r=-0.061. Namely, the correlation factors are less from 0.10, in other words the pairs are practically independent. Finally, the seismic target-displacement at the top of the tower from linear response history analysis are 0.31m for horizontal translational displacement and $2.6 \cdot 10^{-4}$rad for torsional displacement due to eccentric mass at the top of the tower.

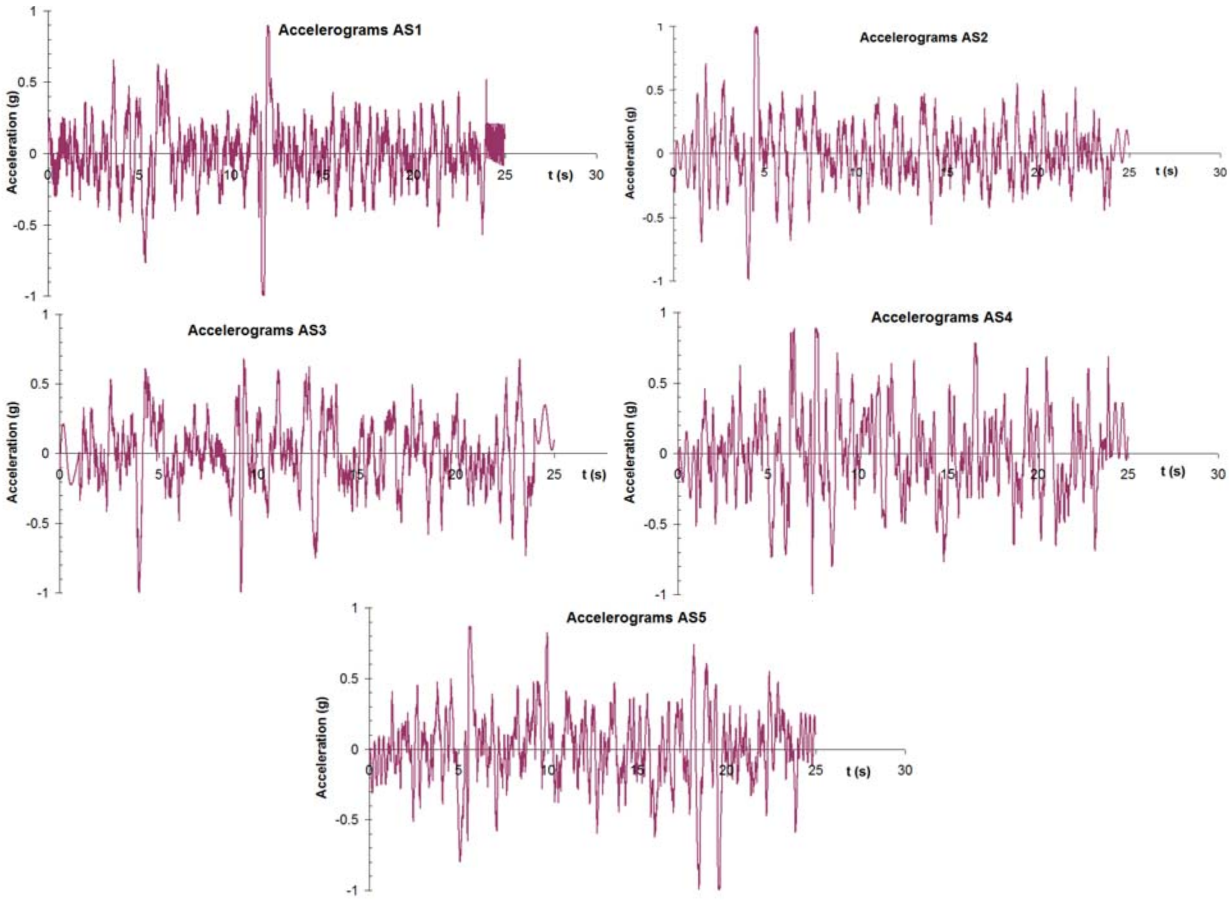

Fig. 6 Five accelerograms normalized in unit PGA

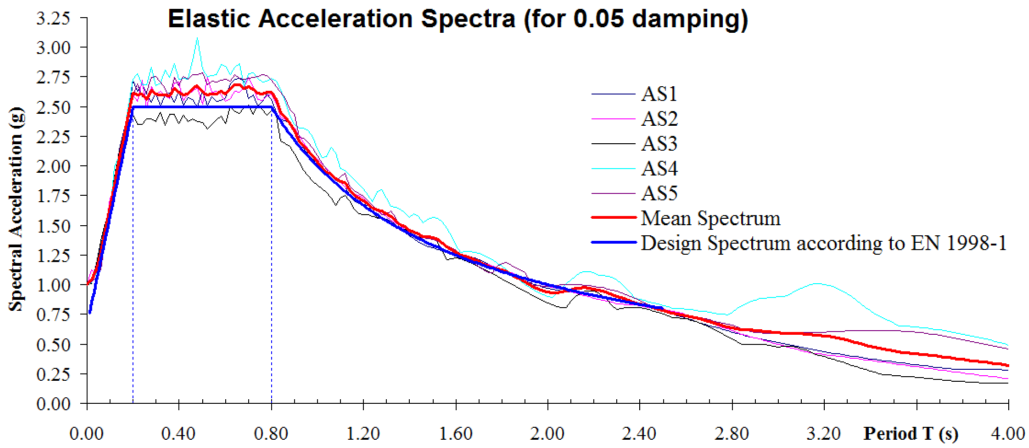

Fig. 7 Elastic acceleration spectra of five accelerograms for 0.05 damping

5. EQUIVALENT STATIC TORSIONAL MOMENT AT THE TOP OF THE TOWER

As it is known from previous relative studies (Makarios & Baniotopoulos, 2013a,b), in order to design the wind energy towers the following two points must be satisfied:

1. According to the paragraph 4.3.1(2) of Eurocode EN 1998-6 either the Response Spectrum Analysis (using several mode shapes until the modal mass reaches the 90% of the total one) or the Response History Analysis is strongly recommended to be applied instead of more simplified methods (cf. e.g. the equivalent static analysis ones). In this sense, the equivalent static method seems not to be acceptable for the seismic analysis of the structures under consideration. For this reason, below, an appropriate variation of the equivalent static method is proposed.
2. Although in the paragraph 4.5 of Eurocode EN 1998-6 this case is not explicitly stated, it would be advisable to apply a loading combination of the earthquake action and the wind loading such as $\pm E \pm 0.5W$, where E and W are the design analysis values for the earthquake and the wind action, respectively.

It is worthy noting that in order to encounter the issue of "fictitious change" of the important first eigen-periods of the tower due to the fact of possible unrealistic assumptions in the formulation of the model, as well as the previously mentioned ambiguities in accuracy, convergence and solution stability, the design spectrum for elastic analysis proposed at the paragraph 3.2.2.4 of EN 1998-1 should be modified as follows for all ground types, so that no reduction of the total seismic base shear to be observed:

$$0 \le T \le T_B \quad : \qquad S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3} \right) \right] \tag{11}$$

$$T_B \le T \le 1.60 \text{ s} \quad : \qquad S_d(T) = a_g \cdot S \cdot \frac{2.5}{q} \tag{12}$$

$$1.60 \text{ s} \leq T \quad : \qquad S_{\mathrm{d}}(T) = a_{\mathrm{g}} \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{1.60}{T}\right] \tag{13}$$

The proposed design acceleration spectrum is almost identical to the Eurocode EN 1998-1 one with only two rather minor different characteristics: (a) the plateau is extended until the period 1.60 s and (b) the characteristic period T_{D} is ignored. This way, the fictitious change of the large tower eigen-periods due to unrealistic assumptions of the model is encountered without amplification of the design earthquake.

In practice, taking into account all the above remarks as well as the results of the linear response history analysis, an equivalent linear oscillator-tower with a concentrated mass at the top can be considered being equal with a half of the distributed mass in elevation plus the concentrated mass of the rotor and blades. Thus:

Weight in elevation: 50%x1422kN=711 kN

Weight of Rotor and blades: 1067 kN

Total weight 1778 kN

Total mass *m*=1778/9.81=181.24t

This oscillator-tower is loaded with equivalent static force *F*:

$$F = m \cdot S_{\mathrm{d}}(T) = m \cdot \left(a_{\mathrm{g}} \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{1.60}{T}\right]\right) = 181.24 \cdot \left(0.16g \cdot 1.35 \cdot \frac{2.5}{1.00} \cdot \left[\frac{1.60}{2.59}\right]\right) = 593.11 \text{ kN}$$

Moreover, consider a torsional moment M_{t} at the top of the oscillator-tower that is numerically equal with the horizontal force *F*, namely M_{t} =593.11 kNm as it results from a pair of forces *F* with unit level-arm. By the simultaneous action of both static loadings, the force *F* and the torsional moment M_{t}, which act at the top of the real wind energy tower, a successful approach according to the results of the linear response history analysis is achieved.

3. CONCLUSIONS

In the present paper, the torsional-translational structural behavior of a wind turbine tower considered as an irregular structure has been examined. An appropriate model of the prototype irregular wind turbine tower by using shell finite elements is herein developed. As is well-known, the action of the higher mode-shapes is utmost important and may become critical in the case that the tower is subjected to strong dynamic loading, as is the wind loading simultaneously excited by a strong seismic motion. Thus, in order to estimate the role of fundamental torsional mode shapes of the above mentioned structure in the final results, seven pairs of appropriately chosen artificial seismic accelerograms that have response acceleration spectra (for ratio damping 0.05) equivalent to elastic acceleration spectra as proposed by Eurocode EN 1998-1, have been used. To this end, using a type of backwards analysis, an equivalent static torsion loading (in the frame of the equivalent static method) has been proposed.

REFERENCES

Baniotopoulos, C.C., Borri, C. and Stathopoulos, T. (2012). *Environmental Wind Engineering and Design of Wind Energy Structures*. Springer Wien, New York.

Baniotopoulos C.C., Lavasas I., Nikolaidis G., Zervas P., Efthimiou E. and Doudoumis I. (2004) On the Design of the Prototype of Wind Turbine Towers. *Proceedings COST C14 International Conference on the impact of wind and storm on city life and built environment*, Brussels, Belgium, 5-7 May: F.7.1-10.

Bayraktar A., Altunişik A., Sevim B. and Türker T. (2011). "Seismic Response of a Historical Masonry Minaret using a Finite Element Model Updated with Operational Modal Testing". *Journal of Vibration and Control, January 2011; vol. 17, 1: pp. 129-149.*

Beer E.P. and Johnston Jr E.R. (1992). *Mechanics of Materials*. 2nd Edition, McGraw Hill.

EN 1998-1. (2004). *Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings*. European Committee for Standardization, Brussels.

EN 1998-6 (2005). *Eurocode 8: Design of structures for earthquake resistance – Part 6: Towers, masts and chimneys*. European Committee for Standardization, Brussels.

Gasch R. and Twele J.(2012). *Wind Energy Plants*. Springer, Heidelberg, New York.

Kessentini S., Choura S., Najar F. and Franchek M.A. (2010). "Modeling and Dynamics of a Horizontal Axis Wind Turbine". *Journal of Vibration and Control, November 2010; vol. 16, 13: pp. 2001-2021.*

Lavassas I., Nikolaidis G., Zervas P., Efthimiou E., Doudoumis I. and Baniotopoulos C.C. (2003). "Analysis and Design of the Prototype of an 1 MW Steel Wind Turbine Tower". *Engineering Structures Journal*. 25: 1097-1106.

Makarios T. and Baniotopoulos C.C. (2013a). "Wind Energy Structures: Modal Analysis by the Continuous Model Approach". *Journal of Vibration and Control*, doi: 10.1177/1077546312463761. (in press, on line October 31, 2012)

Makarios T. and Baniotopoulos C.C. (2013b). "Wind turbine towers with partially fixed foundation: Modal Analysis by the Continuous Model Approach". *Journal of Vibration and Control*, (under review).

Moutsopoulou A, Pouliezos A. and Stavroulakis G.E. (2009): Simulation and modelling using robust control of wind induced vibration on smart beams. *Proceedings COMPDYN 2009 2nd International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*, Rhodos, Greece, 22-24 June: 582-594.

Stavroulakis G.E., Foutsitzi G., Hadjigeorgiou E., Marinova D.G, Zacharenakis E.C. and Baniotopoulos C.C. (2006). "Numerical Experiments on Smart Beams and Plates", *Proceedings smart 2006, Conference*, Rome, 26-28.10.2006, 218-236.