On the torsional-translational response of wind turbine structures

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Abstract:	In the present paper, the torsional-translational response of a prototype wind turbine tower considered as an irregular structure is examined. As a matter of fact a plethora of fatal failures of wind turbines towers have occurred due to torsional dynamic actions. An effective numerical model of the prototype irregular wind turbine tower is herein developed which has been verified by the application of the continuous model method for both the cases of fixed and partially fixed foundations. As known, higher eigenshapes strongly affect the structural response and may become critical in the case that the tower is subjected to strong dynamic loading, as is wind loading, whereas the tower is simultaneously excited by a strong seismic motion. In order to estimate the role of the fundamental torsional mode shapes of the above mentioned structure in the overall structural response of the tower, three pairs of appropriately artificial seismic accelerograms having response acceleration spectra (for equivalent viscous damping ratio 0.03) equivalent to the Eurocode elastic acceleration spectra are used. To this end, applying a type of backwards analysis, an equivalent dynamic or static torsion loading is defined.
Response to Reviewers:	Reviewer 1 Torsional loads determination-Paragraph required The paper addresses the torsional response in a wind turbine and the authors claim that such behaviour is a cause of collapse. While the analysis carried out in the paper shows a method for the estimation of the torsional response, it is not clear how the authors obtained the torsional excitation loads. Considering that an eccentricity of 8m exists is not clear at all for the reader, why 0.1H was assumed for this eccentricity? A paragraph justifying this claim is necessary, not to mention that a study was carried out with no evidence. Authors' Answer: A paragraph highlighted in yellow has been added in chapter 4. of the paper.

Spectrum
A spectrum showing the excitation load is important. Earthquakes excite higher frequencies, while wind loads excite lower frequencies. It is not reasonable to synthesis a load from one spectrum and claim it is a representative for both wind and earthquake loads.

Authors' Answer: A paragraph highlighted in green has been added as a last paragraph of the chapter 4 of the paper.

Additions to References
The literature review will benefit from additional references, including those mentioning the along-wind, cross-wind and torsional responses combined all together, for response estimation in high-rise structures. The following papers should be cited:
Aly, A.M. (2014), "Influence of Turbulence, Orientation, and Site Configuration on the Response of Buildings to Extreme Wind," The Scientific World Journal, 2014, Article ID 178465, 15 pages.
Aly, A.M. (2013), "Pressure Integration Technique for Predicting Wind-Induced Response in High-Rise Buildings," Alexandria Engineering Journal, Elsevier, 52(4), 717-731
Rosa, L., Tomasini, G., Zasso, A. and Aly, A.M. (2012), "Wind-induced dynamics and loads in a prismatic slender building: modal approach based on unsteady pressure measures," Journal of Wind Engineering and Industrial Aerodynamics, 107-108, 118-130.

Authors' Answer: All abovementioned papers have been added to the References List of the paper.

Reviewer 2
English editing
The paper is technically sound, but its English needs considerable editing: there are many grammatical errors, some terms used (even technical) are wrong, etc. Presumably, the third author (who is teaching at a British University) is in a position to carry out a thorough editing of the text.

Authors' Answer: The grammatical errors of the paper have been corrected and the text was proofread .

Spectrum+ Amplitude part of the envelope
From the technical point of view, the very long acceleration-controlled range of the spectrum (extending up to 1.6 sec) is not easy to justify or explain. The same holds for the very long constant-amplitude part of the envelope (modulating function) chosen for the generation of the artificial records. The authors refer to other papers of theirs concerning these aspects; however, this is not convincing. Some explanation is needed.

Authors' Answer: A paragraph highlighted in magenta has been added.

Additional references related to the topic?
Finally, almost all references listed come from the authors' own work. If nobody else has produced any work worthy of quoting and discussing in the paper, is then the topic so important as the authors claim?

Authors' Answer: In the Reference List some relevant papers have been added (cf. e.g. papers [17], [18] and [19] have been added).

To conclude with, the authors would like once more to sincerely thank both reviewers for their valuable comments.

Sincerely yours

	Triantafyllos MAKARIOS

On the torsional-translational response of wind turbine structures

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Abstract

In the present paper, the torsional-translational response of a prototype wind turbine tower considered as an irregular structure is studied. As a matter of fact a plethora of wind turbine towers have collapsed during the last decades due to torsional dynamic actions. An effective numerical model of the prototype irregular wind turbine tower is herein developed which has been verified by the application of the continuous model method considering both a fixed and a partially fixed foundation. As known, the higher eigenmodes of the tower strongly affect the structural response and may become critical in the case that the tower is subjected to strong dynamic loading, as is e.g. wind loading, when simultaneously excited by a strong seismic motion. In order to estimate the role of the fundamental torsional modes of the above mentioned structure in its overall structural response, three pairs of appropriately selected artificial seismic accelerograms having response acceleration spectra (for equivalent viscous damping ratio 0.03) equivalent to the Eurocode elastic acceleration spectra are used and then, applying a type of backwards analysis, an equivalent dynamic or static torsion loading is defined.

Keywords: Wind turbine structures, torsional-translational response, numerical analysis.

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1. Introduction

At Searsburg, a town in north-eastern United States, a turbine of the wind farm there collapsed in 2008 under extreme wind conditions. During this event, one of the turbine blades hit the base destabilizing the tower and leading the nacelle and rotor assembly to crash on the ground [1] (**Fig.1**). The collapse mode of the wind turbine tower was identical to the standard torsional collapse mode about the vertical axis of the tower that theoretically is due to strong wind pressure and related aeroelastic fluttering in combination to a significant seismic excitation. Due to the aerodynamic loading, aeroelastic phenomena leading to excessive elastic deformations often appear leading in certain cases to structure's collapse, cf. e.g. the reference Tacoma Narrows Bridge collapse in 1940. A plethora of studies on the aeroelastic behaviour of wind turbines have been published the last years [2-5]. As the height of the wind turbine towers the last decades significantly increased up to 200m, the length of the rotor blades also significantly increased, a fact leading to strong aeroelastic effects (e.g. strong torsional vibrations on the tower or even collapse of the blades themselves). Specific structural design guidelines against torsional vibrations are in general not provided by modern Structural Codes and therefore, tall wind turbine towers could be considered as exposed to this collapse mode.

Wind energy structures are expected in the near future to correspond to a significant part of energy produced by renewable energy systems (cf. e.g. [6-7]) and therefore, the need to further enhance the energy systems applications in terms of efficiency is indispensable [8]. A plethora of Aeolian parks with numerous wind energy towers are nowadays under erection or planned to be erected; most of these towers are steel tubes with reduced thickness along their heights. Although the structural design of such towers could be considered as a rather simple task, the variety of irregularities appearing due to the blades shape, the concentrated mass on the top, the application of the dynamic loadings, the aeroelasticity phenomena and the peculiarities of their foundation, requests their analysis and design to be performed in the most meticulous way [9]. To this end, several significant design issues as are local buckling analysis of the shell structure, stress concentration states around the opening and tower modal analysis have to be thoroughly examined [10]. A modal analysis of a prototype tubular tower with fully fixed foundation has been recently carried out by applying the well-known continuous model approach, and the role of a partially fixed foundation due to uplift that likely leads to overturning has been studied by the same method [11], [12]. It is obvious that the complexity of the aforementioned structural analysis issues at hand requests sophisticated and innovative treatment (cf. e.g. [13-14]). Advanced mathematical models for the analysis of the structural response of horizontal axis wind turbines with flexible tower and blades were developed by Kessentini *et al*. [15], where the eigenvalue problem was treated both analytically and numerically by applying the differential quadrature method. The use of the finite element method for the tower analysis in combination with the identification of the tower dynamic characteristics via ambient vibrations is nowadays considered as a standard technique for the tower structural behavior modeling [16]. Moreover, advanced procedures and techniques have been examined about the extreme wind loadings on the towers [17-19]. Recently, in order to encounter the torsional behavior of the wind turbine towers an appropriate backwards analysis has been proposed by Makarios & Baniotopoulos [20] that leads to the calculation of the equivalent static torsion loading at the top of the tower under consideration. In the next paragraphs the latter being an advanced torsion analysis of a prototype wind turbine tower is in details presented and illustrated by means of a numerical example.

2. Mathematic formulation of torsional behavior

2.1 Torsional behavior of a circular cantilever

In order to theoretically examine the torsional behavior of a steel tubular wind energy tower AB, the Technical Torsion Theory is applied to a cylinder-cantilever with section radius R loaded at its top by a torsional moment M_t [21] (**Fig. 2**). Considering that each section of the cantilever AB is loaded by the same torsional moment M_t, the relative rotation of this cantilever is constant along all its length L. Considering an infinitesimal element dz, where the line PΛ becomes PΛ′ with the shear deformation γ_R being the angle and ignoring the second order infinitesimals, the following form is obtained:

$$\gamma_R = \frac{\Lambda\Lambda'}{\mathrm{d}z} = \frac{R \cdot \mathrm{d}\varphi}{\mathrm{d}z} \tag{1}$$

Thus, the shear stress τ_R yields:

$$\tau_R = G \cdot \gamma_R = \frac{\Lambda\Lambda'}{\mathrm{d}z} = G \cdot \frac{R \cdot \mathrm{d}\varphi}{\mathrm{d}z} \tag{2}$$

where G is the shear modulus, namely $G = \frac{E}{2(1+v)}$, where E is the modulus of elasticity and v is the Poisson's ratio.

In addition, the shear stress component $\tau(r)$ is perpendicular to the radius R , while its value along the radius R for $r \leq R$ is given as (Fig. 3):

$$\tau(r) = G \cdot \frac{r \cdot \mathrm{d}\varphi}{\mathrm{d}z} \tag{3}$$

By means of the equilibrium equations, the total moment of the internal shear stresses $\tau(r)$ of a section should be equal to the external torsional moment $, and\ t$hus,

$$\int_{r=0}^{r=R} r \cdot [\tau(r) \cdot (2\pi \cdot r \cdot \mathrm{d}r)] = M_t \tag{4}$$

Inserting Eq.(3) into Eq.(4) , the following form is obtained:

$$\int_{r=0}^{r=R} r \cdot \left[G \cdot \frac{r \cdot \mathrm{d}\varphi}{\mathrm{d}z} \cdot (2\pi \cdot r \cdot \mathrm{d}r)\right] = M_t \tag{5}$$

Thus, Eq.(5) becomes:

$$2\pi \cdot G \cdot \frac{R^4}{4} \cdot \frac{\mathrm{d}\varphi}{\mathrm{d}z} = M_t \qquad \Rightarrow \tag{6}$$

$$\frac{\mathrm{d}\varphi}{\mathrm{d}z} = \frac{M_t}{G \cdot I_\mathrm{p}} \tag{7}$$

where I_p is the polar moment of inertia I_p (namely, $I_\mathrm{p} = \pi \cdot \frac{R^4}{4}$ for the case of a circular section).

2.2 Undamped torsional response 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 $\mu_\mathrm{t}(t,z)$. The section polar moment of inertia $\bar{I}_\mathrm{p}(z)$, as well as the section mass moment of inertia $\bar{J}_\mathrm{m}(z)$ are functions that depend on the height *z*.

Consider an infinitesimal element of the cantilever of length dz, where the internal torsional moments on the element are as depicted in **Fig. 4**. On this infinitesimal element the inertia torsional moment $\bar{M}_{\mathrm{t},a} = -\bar{J}_\mathrm{m}(z) \cdot \frac{\partial^2 \varphi(z,t)}{\partial t^2}$ is formulated by applying the D'Alembert's principle where $M_\mathrm{t}(z,t)$ is the torsional moment according to the well-known aforementioned Technical Torsion Theory. Furthermore, it is known that $\bar{J}_\mathrm{m}(z) = I_\mathrm{p}(z) \cdot \frac{\bar{m}(z)}{A(z)}$, where $\bar{m}(z)$ is the mass per unit length in elevation and $A(z)$ is the area of the section at level *z*. Equilibrium of vectors of torsional moments on the differential element in *z*-direction yields:

$$-M_\mathrm{t}(z,t) + \mu_\mathrm{t}(t,z) \cdot \mathrm{d}z + \left[M_\mathrm{t}(z,t) + \frac{\partial M_\mathrm{t}(z,t)}{\partial z} \cdot \mathrm{d}z\right] - \bar{J}_\mathrm{m}(z) \cdot \frac{\partial^2 \varphi(z,t)}{\partial t^2} \cdot \mathrm{d}z = 0 \tag{8}$$

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

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

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

Eq.(10) is a equation which can be effectively treated numerically. In order to develop an efficient model to simulate the structural response of the prototype wind energy tower at hand, the continuous model method has been applied for both the fully fixed and the partially fixed foundation case [12], [20]. By means of this method, two effective numerical models have been respectively developed which in the present paper were appropriately modified to examine the torsional behavior of the prototype wind energy tower.

3. FEM Modeling and Shell modal analysis

The prototype of the thin-walled tower at hand supporting a 2 MW wind turbine is considered. 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 up to the top where the tower diameter is 3.0 m. Shell thickness varies linearly from 30 mm at the bottom to 12 mm at the top. The steel quality of the structure is S355, while the modulus of elasticity E=210 GPa. Moreover, the self-weight of the tower is 1480 kN and the blade self-weight is $W_0 = 1067\ kN$, 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 $W_0 = 1067\ kN$ is located 0.50 m above the upper section of the tower. The model of the wind energy tower using shell finite elements and the first mode shapes are also shown in Fig. 5. It is clear that the activation of the 3rd, 4th and 6th mode shape of the tower 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 (**Fig. 1**). Thus, the critical frequencies of the modal analysis are:

$$f_3 = \frac{1}{T_3} = \frac{1}{0.135} = 7.41\ \text{Hz}\ , \quad f_4 = \frac{1}{T_4} = \frac{1}{0.131} = 7.63\ \text{Hz}\,, \qquad f_6 = \frac{1}{T_6} = \frac{1}{0.09} = 11.11\ \text{Hz}$$

4. Time function of the torsional moment loading

In order to obtain the torsional behaviour of a wind energy tower, a static loading that leads to the development of torsional moments $\mu_t(z)$ (around z-axis) with continuous distribution in elevation has to be inserted on the tower (one with positive sign and another one with negative sign). The base torsional moment M_{t} of the above mentioned continuous torsional moment $\mu_{\mathrm{t}}(z)$ of the tower is calculated by its seismic base shear V_o multiplied by an accidental eccentricity e_a. The respective parametric analysis performed concluded to an accidental eccentricity equal to $e_a = \pm\, 0.10 \cdot H_{\mathrm{tot}}$, where the total height of the wind turbine tower is H_{tot}.

A critical assessment of the contribution of the torsional mode-shapes to the overall analysis results of a wind turbine tower is presented in [20]. The analysis can be performed for the tower subject to a certain concentrated torsion about the z axis at the top having a value equal to the seismic base shear. In this analysis, the magnitude of the steel yield stress and the tower torsional deformation at the yield state have been both taken into account. By means of the proposed iterative procedure, the magnitude of the concentrated torsional moment has been approximated; the latter is combined to the rest design actions so that the above mentioned torsional deformation at the yield state to be reached. Although this concentrated loading approach leads to satisfactory results for towers without or with small number of internal stiffening rings, this analysis does not generally provide the designer with appropriate results, in particular when the response history analysis is applied. For this reason, in the present study an amended method is proposed considering the loading along the height of the tower distributed $\mu_{\mathrm{t}}(z)$ and having a form related to the time function $f(t)$. By means of this approach the average value of the tower accidental eccentricity is $e_a = \pm\, 0.10 \cdot H_{\mathrm{tot}}$. Then, the dynamic torsional loading $\mu_{\mathrm{t}}(z, t)$ is coupled to the seismic time-history excitation leading in all cases to realistic results.

It is always clear that $\mu_{\mathrm{t}}(z) = M_{\mathrm{t}}/H_{\mathrm{tot}}$. Furthermore, in the case of a response history analysis, the previous base torsional moment $M_{\mathrm{t}}(t)$ has the following time-history form:

$$M_{\mathrm{t}}(t) = \pm\, e_a \cdot V_o \cdot f(t) \tag{11}$$

where the $V_o = m_{\mathrm{tot}} \cdot S_{\mathrm{e}}$ is the seismic base shear according to the modified design acceleration spectrum (**Fig.6**) as proposed for wind turbine towershaving the same form for each ground category respectively [11-12, 20]:

$$0 \le T \le T_{\mathrm{B}} = 0.20\mathrm{s} \qquad S_{\mathrm{e}}(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_{\mathrm{B}}} \cdot (\eta \cdot 2.5 - 1)\right] \tag{12}$$

$$T_{\mathrm{B}} < T \le T_{\mathrm{C}} = 1.60\mathrm{s} \qquad S_{\mathrm{e}}(T) = a_g \cdot S \cdot \eta \cdot 2.5 \tag{13}$$

$$1.60s < T \qquad S_{\mathrm{e}}(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_{\mathrm{C}}}{T} \tag{14}$$

where a_g is the effective acceleration of the seismic hazard zone (in g-units), the damping coefficient given by $\eta = \sqrt{\frac{0.10}{0.05+\xi_{\mathrm{ef}}}}$, with ξ_{ef} the equivalent ratio viscous damping referring to the critical damping, S the factor of the ground category according to EN 1998-1 and m_{tot} the total mass of the wind turbine tower with its blades and rotor. It is worthy to note that the extension of the spectrum plateau up to 1.6s without increase of the seismic risk of the area is proposed as a means to cover any discrepancies and erroneous assumptions in the development of the model and the respective tower eigenfrequencies (and therefore, to take into account the respective resonance phenomena from the external seismic excitation and the first tower eigenmodes) [11-12, 20].

According to the previous analysis, the continuous time-torsional moments $\mu_{\mathrm{t}}(z,t)$ in elevation is (**Fig.4**):

$$\mu_{\mathrm{t}}(z,t) = M_{\mathrm{t}}(t)/H_{\mathrm{tot}} \tag{15}$$

The proposed design acceleration spectrum is almost identical to the one given by Eurocode EN 1998-1 with only two rather minor different characteristics: (a) the plateau is extended up to the period 1.60 s and (b) the characteristic period T_{D} of EN 1998-1 is ignored [22]. This way, the fictitious change of the large tower eigenperiods due to unrealistic assumptions of the model is encountered without amplification of the design earthquake level [20].

With reference to the previous time function $f(t)$, a parametric analysis has been carried out where stresses and deformations due to the continuous torsional moments in elevation have been calculated. From the magnitude of these stresses in combination with the critical ones causing damage and collapse (**Fig.1**), the function $f(t)$ can be assessed having the following characteristics:

a) a total duration 25s at least

b) a form similar to the one of **Fig.7**

c) function (t) is described as follows:

$$f(t) = 0.40t \quad \text{for} \quad t_1 = 0.00 \leq t \leq t_2 = 2.50\text{s} \tag{16}$$

$$f(t) = 1.00 \quad \text{for} \quad t_2 = 2.50 < t \leq t_3 = 22.50\text{s} \tag{17}$$

$$f(t) = 1.00 - 0.40 \cdot (t - t_3) \qquad \text{for} \quad t_3 = 22.50 < t \leq t_4 = 25.00\text{s} \tag{18}$$

d) for the seismic analysis, the response history analysis must be carried out using appropriately selected accelerograms simultaneously for the torsional moments $\mu_\mathrm{t}(z)$ and the wind loading. In the case that these accelerograms have duration of strong ground motion greater than 20s, then $t_3 - t_2$ must be equal to the duration of strong ground motion, since the strong ground motion has to be always located into the time-window $t_3 - t_2$.

It would be advisable to apply a loading combination of the earthquake action and the wind loading such as ±*E*±0.5*W*, where *E* and *W* are the design analysis values for the earthquake and the wind action, respectively, as in the regions where wind turbine towers are constructed there is always significant Aeolian potential [11-12,20]. Obviously the seismic spectrum using high frequencies cannot be in general applied to simulate wind action that is characterised by low frequencies. This is the reason why in the present study the frequency of application of the torsional dynamic loading is defined by means of the time function $f(t)$, whistl the seismic spectrum is used only for the assessment of the magnitude of $\mu_\mathrm{t}(z,t)$ when both earthquake and wind are acting, i.e. load combination ($\pm E \pm 0.5W$).

5. Numerical application

Applying the previously presented approach and taking into account the results of the linear response history analysis, an equivalent linear oscillator-tower with a concentrated mass at the top is first 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

Ground Category: D according to EN 1998-1

Equivalent viscous damping ratio: $\xi_{\mathrm{ef}} = 0.03$

Fundamental period: $T_1 = 2.59\mathrm{s}$ from **Fig. 5**.

Thus, $\eta = \sqrt{\frac{0.10}{0.05+\xi_{\mathrm{ef}}}} = \sqrt{\frac{0.10}{0.05+0.03}}$ =1.12

This oscillator-tower is loaded with equivalent static force V_o:

$$V_o = m_{\mathrm{tot}} \cdot \left(a_g \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_{\mathrm{C}}}{T}\right) = 181.24 \cdot \left(0.16g \cdot 1.35 \cdot 1.12 \cdot 2.5 \cdot \frac{1.60}{2.59}\right) = 664.29\ \mathrm{kN}$$

The accidental eccentricity of the tower is:

$$e_a = \pm\, 0.10 \cdot H_{\mathrm{tot}} = \pm\, 0.10 \cdot 80.00 = \pm\, 8.00\ \mathrm{m}$$

Therefore, the torsional moment $M_t(t)$ is:

$$M_{\mathrm{t}}(t) = \pm\, e_a \cdot V_o \cdot f(t) = \pm\, 8.00 \cdot 664.29 \cdot f(t) = \pm\, 5314.32 \cdot f(t)$$

where $f(t)$ is taken from **Fig.7**.

The uniform torsional-moment $\mu_{\mathrm{t}}(z,t)$ in elevation is calculated as follows:

$$\mu_{\mathrm{t}}(z,t) = M_{\mathrm{t}}(t)/H_{\mathrm{tot}} = \pm 5314.32 \cdot f(t)/80.00 = 66.43 \cdot f(t) \quad \mathrm{kNm/m}$$

In the general case, considering the previous torsional moment $\mu_{\mathrm{t}}(z,t)$ along the height of the oscillator-tower with simultaneous action E of the two translational seismic horizontal components (in accelerograms), as well as the 50% of the lateral wind loading W according to the ±E±0.5W combination, the critical combination is obtained.

In order to calculate the seismic response of the prototype wind turbine tower at hand, a linear response history using three appropriately selected accelerograms is applied (**Fig.8**); the latter are compatible with both, the modified spectrum (Fig.9) and the local conditions by the seismic microzonation study where the spectral amplification is taken equal to 3 instead of 2.50, with the time-depended uniform torsional-moment $\mu_{\mathrm{t}}(z,t)$. The artificial accelerograms applied have been developed keeping quality requirements following a recently proposed method [23-25]. In Fig.10 the trace of the response history displacements of the top of the wind turbine tower is depicted, where the extreme displacement occurs at 16.86s [26]. At this moment, the indicative values of the deformations in elevation, as well as the stresses s11, s22 and s12 due to artificial compatible accelerograms and time-depended uniform torsional-moment $\mu_{\mathrm{t}}(z,t)$ are presented (**Fig.11**).

It is worthy to note that the extreme deformations in elevation due to uniform torsional-moment $\mu_{\mathrm{t}}(z,t)$ appear near the diaphragms, while the extreme displacements about the mid-span between two diaphragms (**Fig.11f**). The use of additional diaphragms in elevation (for instance of diaphragms every 10.00m) seems that it would likely contribute to a safer design strategy against torsional collapse of the tower.

5. Conclusions

In the present paper, aeroelasticity phenomena combined to the torsional-translational structural response of a wind turbine tower considered as an irregular structure have been studied. An appropriate model of a prototype wind turbine tower by using shell finite elements is herein developed. The action 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 simultaneously excited by a strong seismic motion. In order to estimate the contribution of the fundamental torsional mode shapes of the tower into the final results, pairs of appropriately chosen artificial seismic accelerograms that have response acceleration spectra (for equivalent viscous damping ratio 0.03 for steel structures) equivalent to elastic acceleration spectra as proposed by Eurocode EN 1998-1, have been selected and applied. Using a type of backwards analysis, an equivalent time-depended uniform torsion loading has been calculated for application on the tower under investigation. According to the present study, the use of additional diaphragms in elevation significantly contributes to a safer design against torsional collapse of the tower.

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7. Figure captions

Figure 1: Torsional Failure of a Wind Energy Tower [1]

Figure 2: Behavior of a regular cylinder subject to a torsional moment M_{t} at the top.

Figure 3: Section at *z*-level

Figure 4: Behavior of a regular tower loaded with torsional moments in elevation.

Figure 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.

Figure 6: Modified horizontal elastic acceleration spectra for towers for each ground category

Figure 7: Time function $\mathrm{f(t)}$ for the dynamic torsional moment $\mathrm{M_t(t)}$

Figure 8: Artificial accelerograms for seismic hazard zone with effective PGA=0.16g and for soil category D, compatible to proposed spectrum for wind energy towers.

Figure 9: Elastic response acceleration spectra of the three artificial accelerograms of Fig. 8

Figure 10: Response history displacements of the tower due to artificial accelerograms (Art1)-(Art2) by SAP2000 [23]

Figure 11: Results of response history analysis using SAP2000: (a) Distribution of stresses s12 at time 16.86s due to time-depended uniform torsional-moment $\mu_t(\mathrm{z,t})$, (b) Distribution of stresses s11 at time 16.86s due to art1 & art2, (c) Distribution of stresses s12 at time 16.86, due to art1 & art2, (d) Distribution of stresses s22 at time 16.86s, due to art1 & art2, (e) Displacements at time 16.86s, due to art1 & art2, (f) Displacements at time 16.86s due to time-depended uniform torsional-moment $\mu_t(\mathrm{z,t})$.c

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Figure

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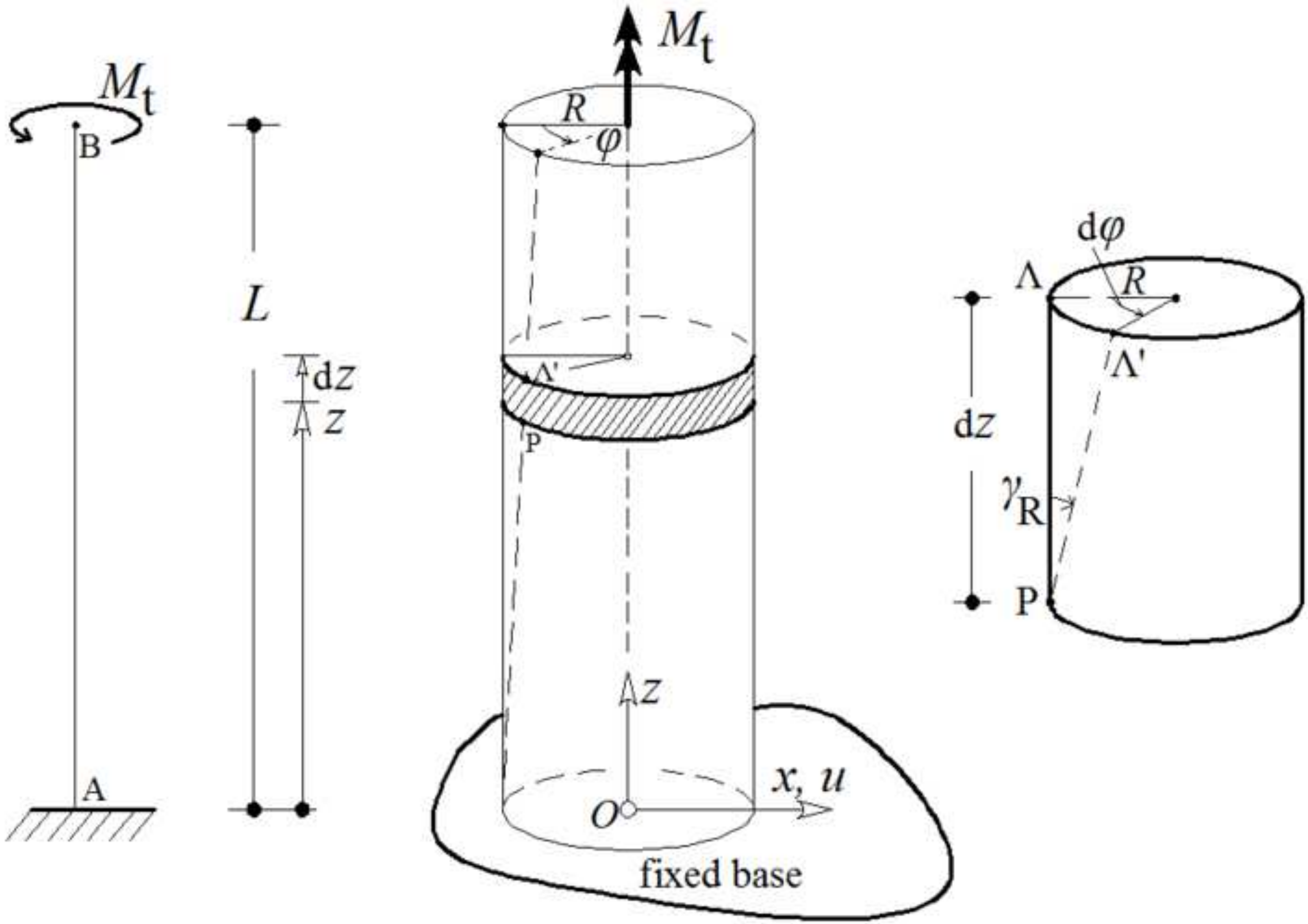

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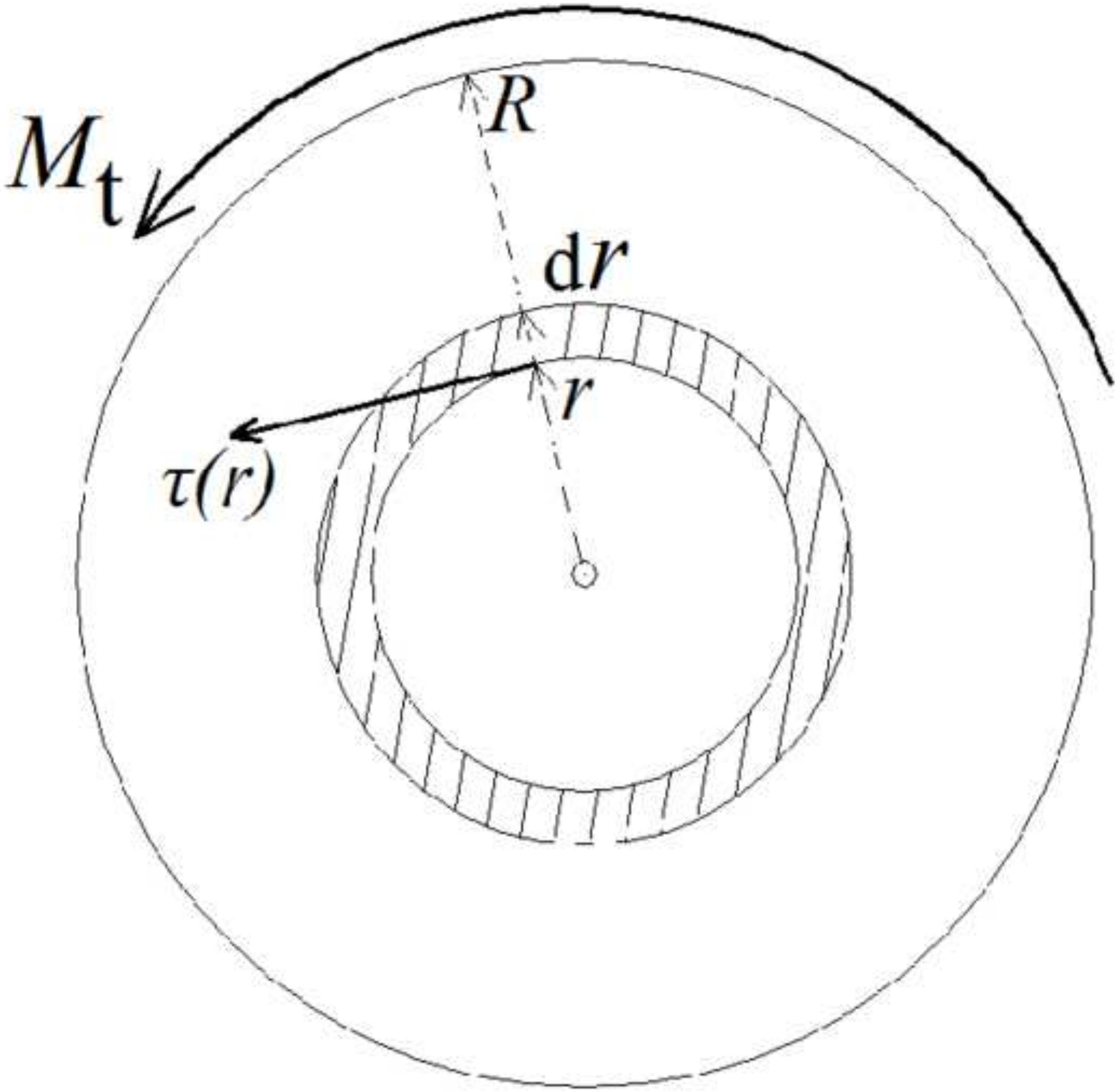

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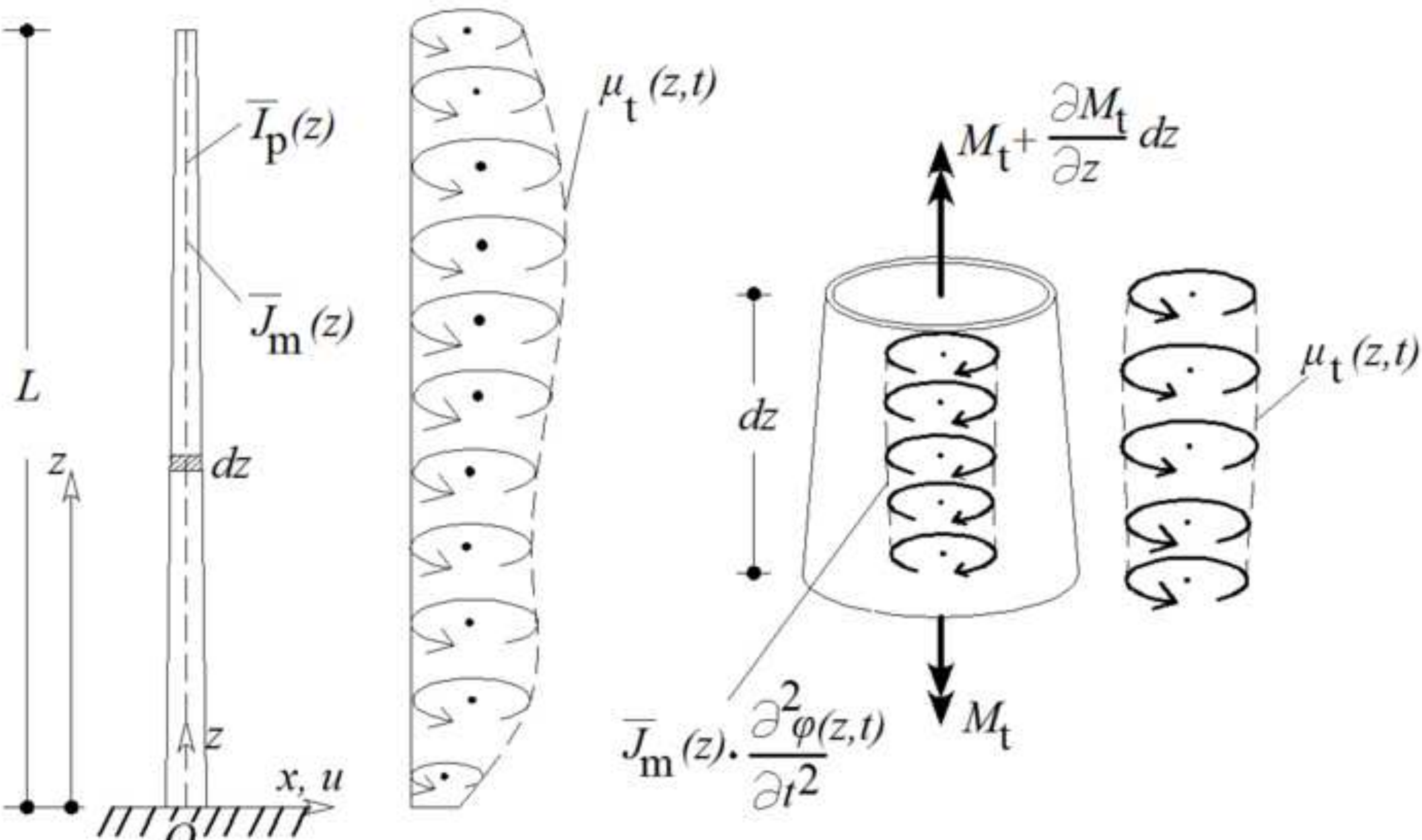

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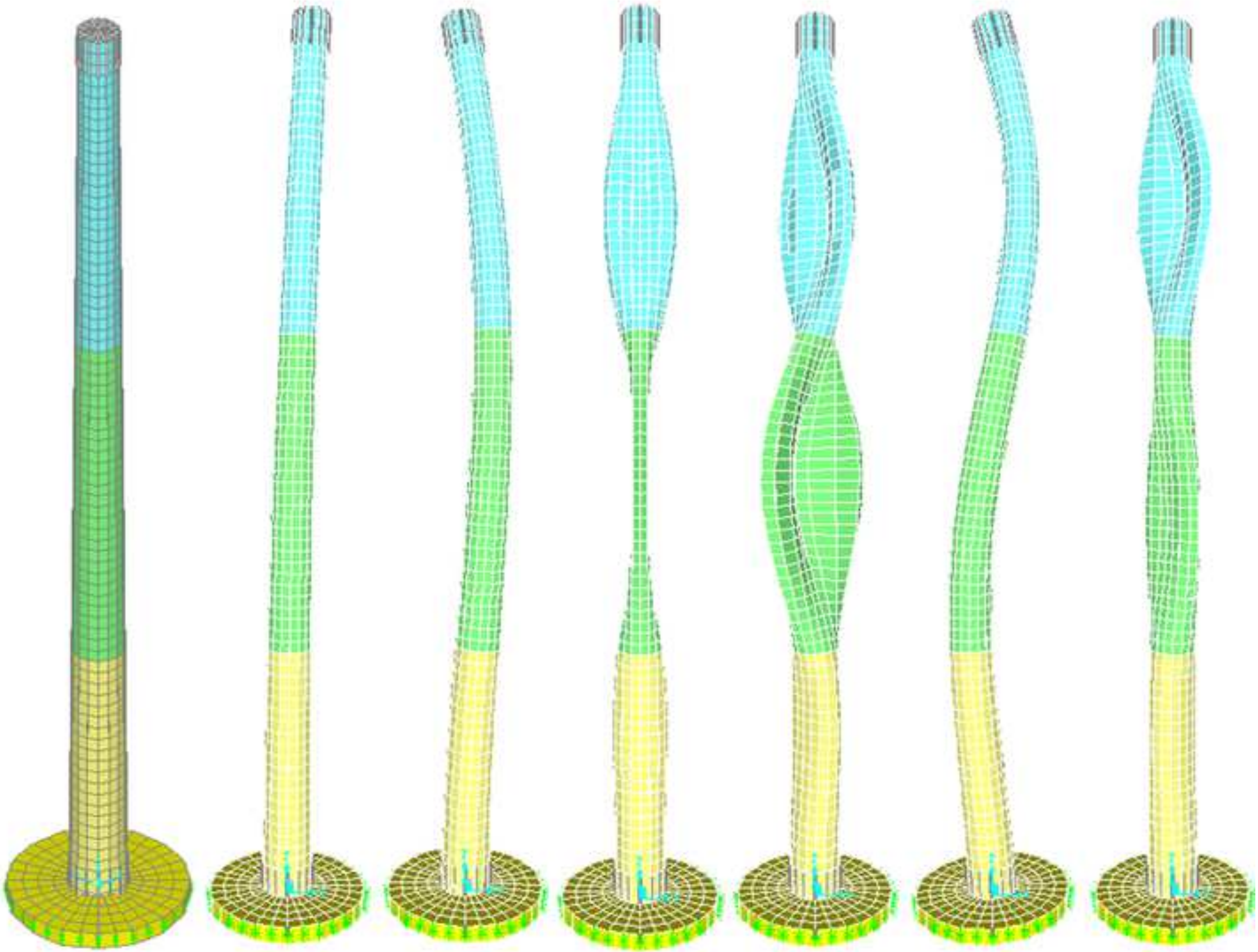

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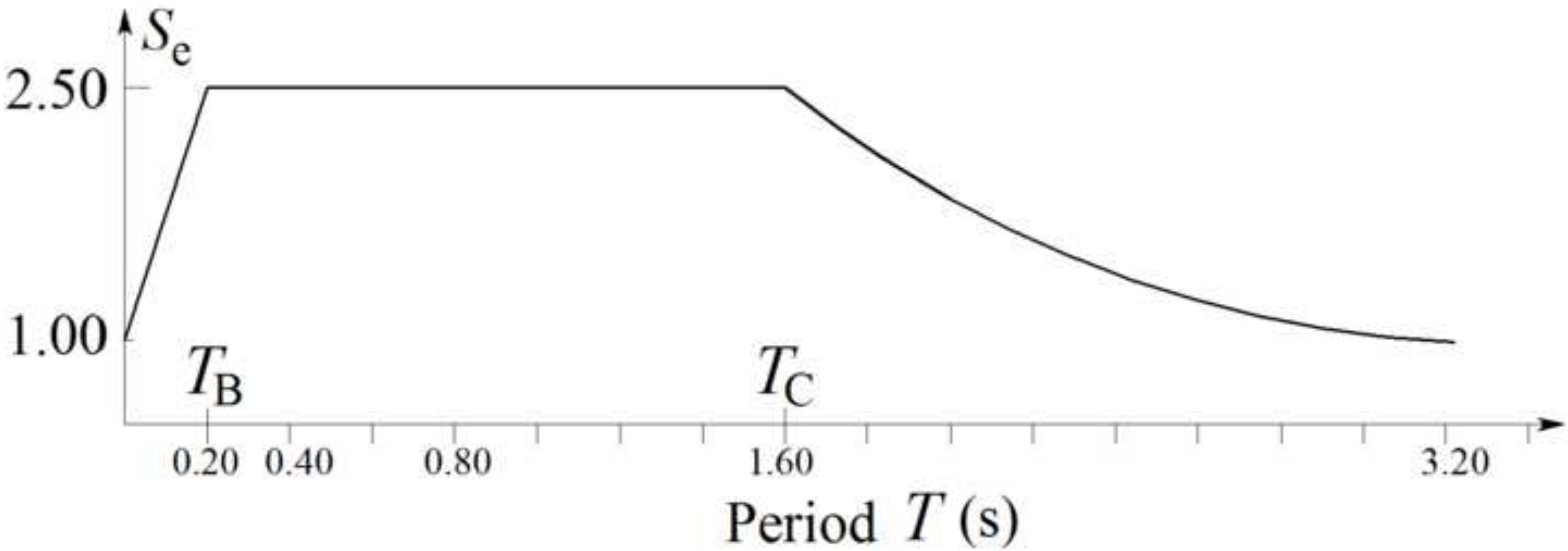

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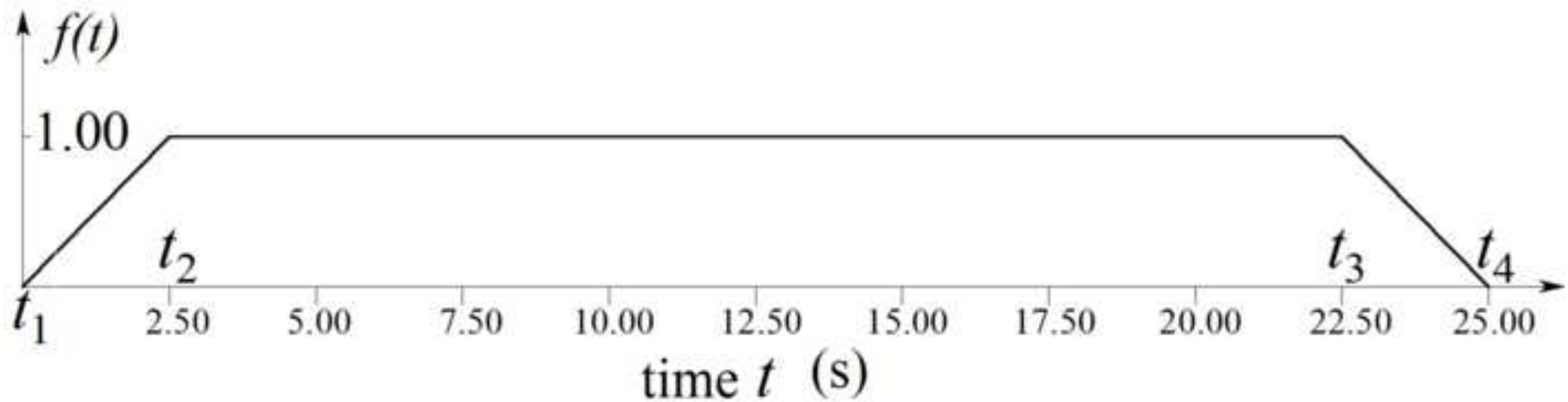

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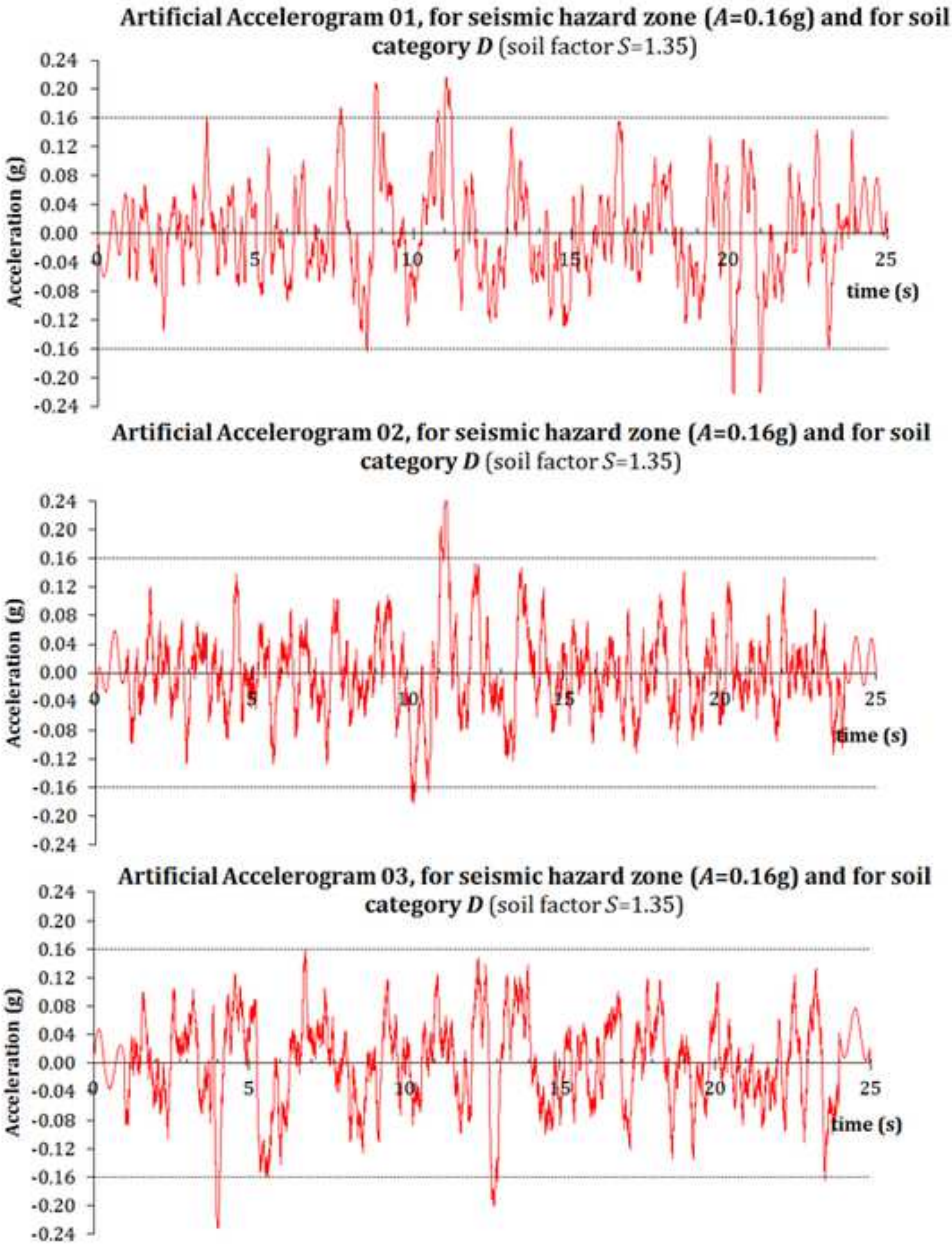

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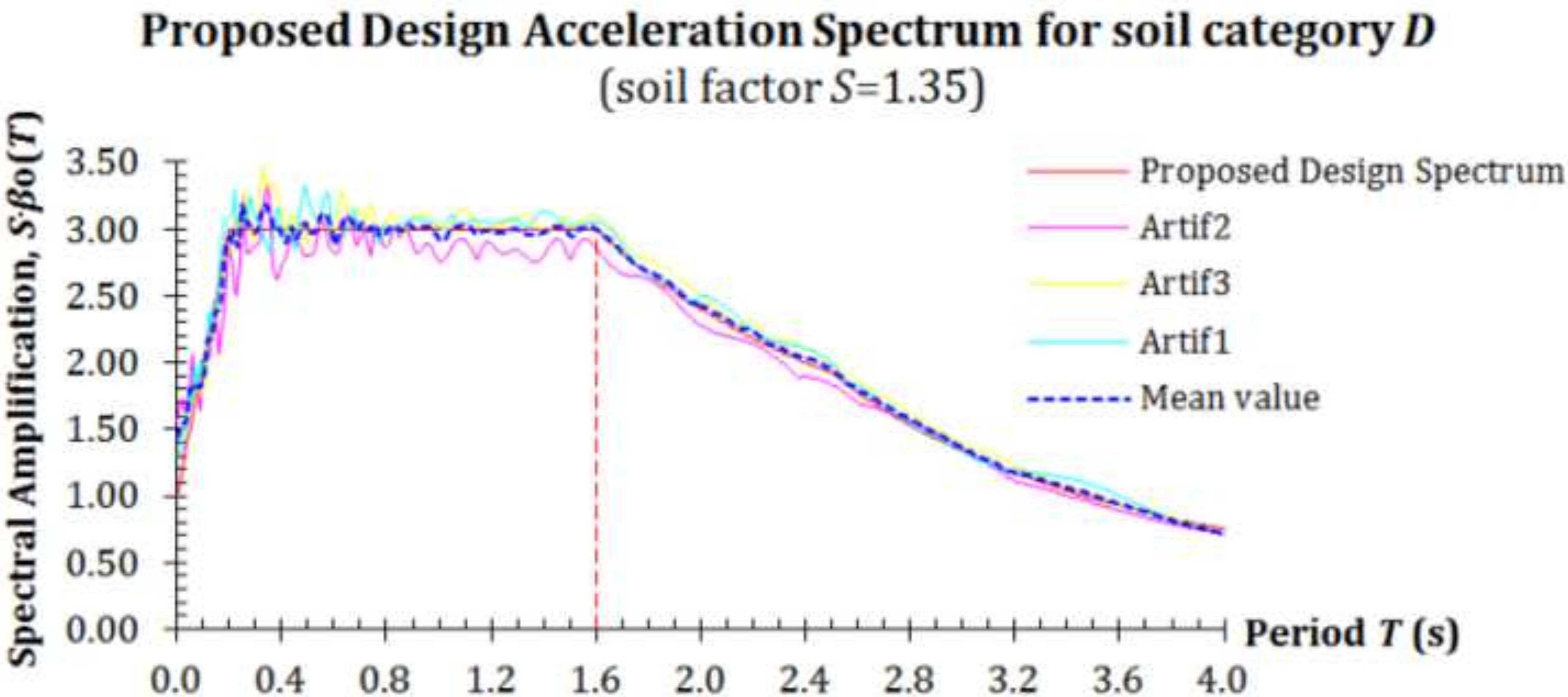

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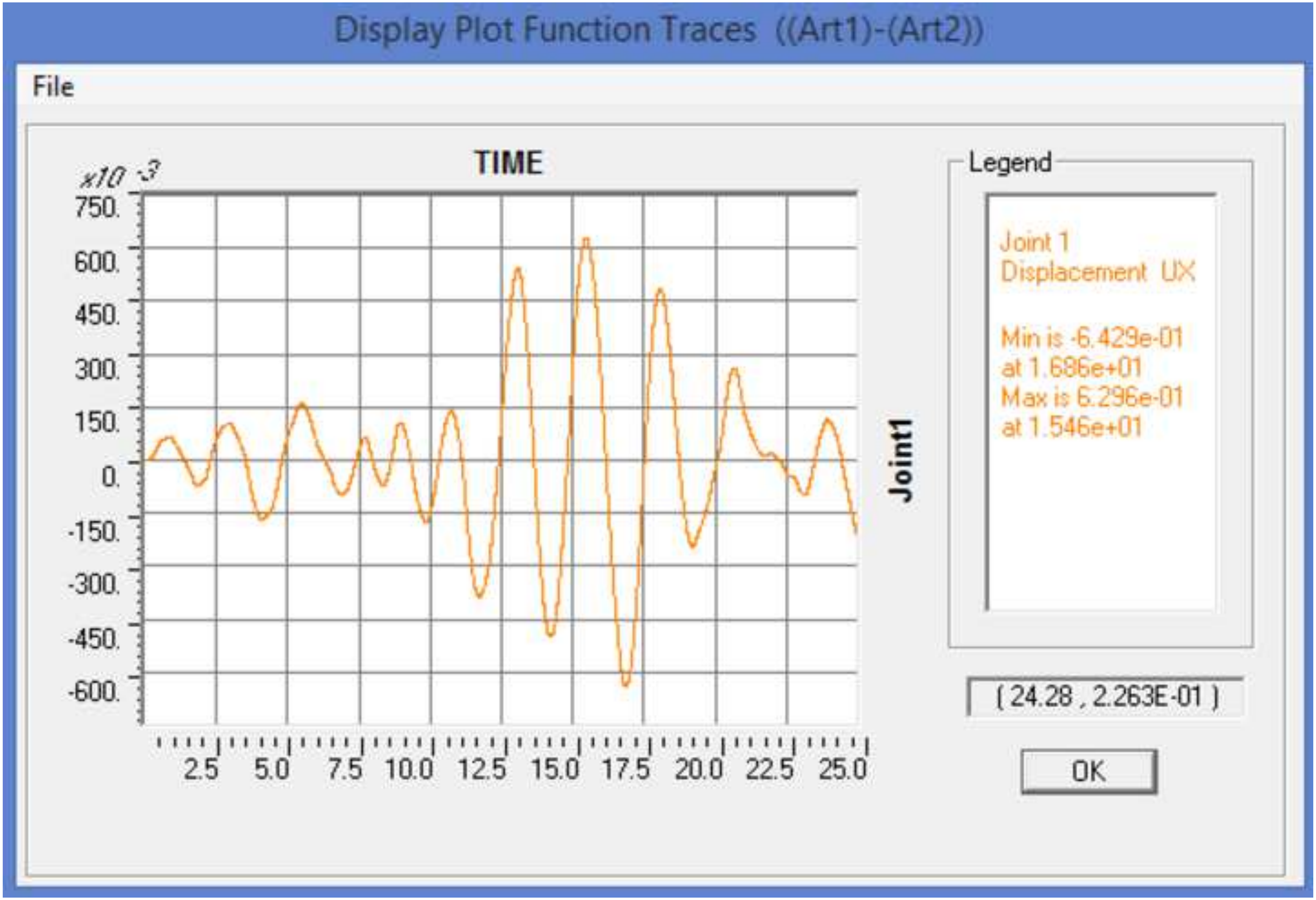

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