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THE DYNAMIC PROPERTIES OF THE ASINELLI TOWER IN BOLOGNA

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Abstract. *The Asinelli Tower, built at the end of the 12th century, is one the main symbol of the town of Bologna and a valuable historical heritage of the Medieval age of the entire Italy. For its structural configuration, the tower appears prone to seismic damages and, therefore, an assessment of its dynamic properties is of primary importance in order to predict the tower seismic response. In the present paper, based on the results of limited material tests, the mechanical and dynamic properties of the tower are analysed through the development of models of increasing complexity. First, models for the evaluation of the main mechanical properties of the masonry (such as the elastic modulus, the Poisson's coefficient, the compressive and shear strengths) are compared in order to validate the experimental results. Then, different structural models of the tower (from simple continuum analytical models to more complex finite element models) are developed. The analytical and numerical results obtained from the different models are finally compared to some recent experimental measurements of the free vibration response of the tower conducted by the INGV (the italian National Institute of Geophysics and Volcanology). The preliminary results indicate that the experimental frequencies are in good agreement with the values obtained from the models.*

1 INTRODUCTION

The Italian historical-monumental heritage is worldwide recognized as one of the richest and various due to the millenary history of the country affected by the influence of different cultures from the ancient Greeks and Romans to the Muslims. Among the various symbols of the Italian monumental heritage, the two medieval towers of Bologna, the Garisenda tower and the Asinelli tower, located in the hearth of the town, are among the most known and attractive ones [1].

The Asinelli Tower, the tallest of the two towers with its 97m height, was built between the XII and XIII century, an historical period characterized by strong political debates, especially between the Church and the Empire. It was commissioned by the one of the most influenced family at the time, the Asinelli family, as a symbol of power and lookout post. During its almost millenary life the tower was subjected to various accidents such as fires (the most destructive is dated 1398 and damaged most of the internal wood structures and the selenitic basement), lightings (in 1754 a lighting caused damages to all the upper part of the tower, for about 30 m of lengths) and earthquakes. The 1399 earthquake caused the failure of the upper part of the tower (the little bell tower) which was reconstructed. It fell down again during the 1505 Appennino Modenese earthquake. Historic reports of that event narrates that also other portion of masonry fell down during that earthquake. Beside the above mentioned natural events, in 1943, during the Second World War, the tower was struck by the effect of a bomb which exploded at few meters from the towers.

It clearly appears that, as a consequence of its long history and continuous interventions after the mentioned catastrophic events, the tower is characterized by large dishomogeneities, part due to the construction technique (leading to the so called “a sacco” masonry, details will be provided in the next section) and part due to different interventions the tower has been subjected during its history. In these cases, from a structural point of view, it is of fundamental importance to evaluate the mechanical and dynamic properties of such monuments, especially in the case of historical towers which, due to their geometrical configuration, are particularly prone to seismic damages.

During the 1990s, the seismicity of Bologna has been analysed within a research program of the italian National Institute of Geophysics and Volcanology (INGV) and a first assessment of the Asinelli Tower risk with respect to seismic events compatible with the seismicity of the area has been carried out (Riva et al. 1998 [2]). In that study, the authors assumed typical values for the material mechanical properties and conclude that the tower stability seems to be not compromised by seismic events of limited seismicity, i.e. the one similar to those of past events occurred in Bologna. However, they suggest in situ and laboratory tests in order to characterize the mechanical properties of the Tower.

In more recent years (end of 1990s), the tower has been subjected to strengthening interventions as a joint project between the Municipality and the University of Bologna. Before the strengthening, in situ and laboratory tests were performed to characterize the mechanical properties of the masonry. After the strengthening, a monitoring system has been installed in order to control the time evolution of the main cracks, the masonry deformation and the tower inclination. Recently, after the 2012 Emilia Earthquake, dynamic tests aimed at identifying the main dynamic properties of the tower have been carried out by the INGV. A report of the experiments is available online [3].

The objective of the present paper is to assess the dynamic behaviour of the tower through the development of several models which are based on the mechanical properties of the materials as obtained from few experimental tests. The numerical results are compared with experimental measurements by the INGV.

2 THE ASINELLI TOWER

The Asinelli Tower is a 97m-high masonry tower with an inclination of 1.7° (corresponding to a overhanging of 2.5m) in the West direction (Figure 1). The tower cross section is approximately square for the whole height with a gradual decrease (almost linear) of the side width from 8.5m at the base to 6.0m at the top, excepting a sudden discontinuity at a height of 34m. The external walls were built using the so called “a sacco” technique (Figure 1): two skins of brick masonry with an internal rubble and mortar fill. The fill is composed of irregular materials including brick fragments and irregular stones bound by aerial mortar. Common solid bricks are used for the outer skins, while the basement is realized with selenitic bricks. The total thickness of the masonry (the two skins plus the internal fill) decreases almost linearly from 3.15m at the base to 0.45m at the top. Three main discontinuities are present at 11.5m, 34.0m and 56.0m. The masonry assemblies are not regular, with variations in both the width of the bricks and the thickness of the mortar (from 1.0cm to 3.0cm). Table 1 provides the main geometrical characteristics along the height of the tower at specific sections.

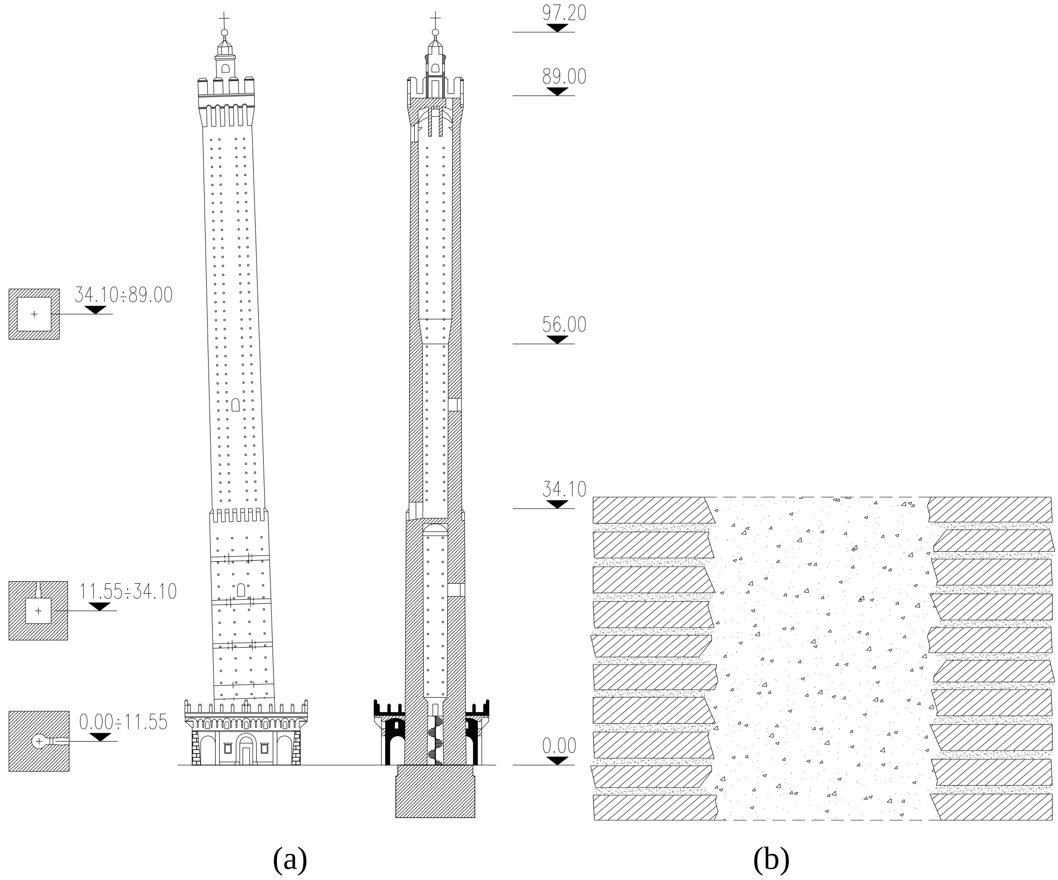

Figure 1: . (a) The tower elevation with the indication of the main discontinuities. (b) A schematic view (vertical cross-section) of the “a sacco” masonry.

2.1 The material properties

In situ and laboratory tests were performed in order to characterize the material properties of the tower. In situ tests included flat jack deformability tests (one compression test and two shear test) according to ASTM 1197C [4] standard and pointing hardness tests (six tests: one on the internal wall and 5 on the external walls) with the hammer pendulum (RILEM

TC127MS D.2, [5]). The flat jack tests were performed on the South side of the Tower at a height of approximately 7m. The compression tests allow to evaluate the masonry Young's modulus (E) and the compression strength (f), while the shear tests allow to estimate the masonry shear strength (f_v). During the pointing hardness tests, the penetration of the drill penetration is measured and used for correlations with the previous mechanical properties. The mean values of the Young's modulus (E_m), the compression strength (f_m) and the shear strength ($f_{v,m}$) of the masonry assemblies and mortar as obtained from the in-situ tests are collected in Table 2.

Table 1: Mean values of the materials mechanical properties.

Material	E_m (MPa)	f_m (MPa)	$f_{v,m}$ (MPa)
Masonry assemblies	3200	4.0	0.7
Mortar	2000	1.9	n.a.

Laboratory tests performed on masonry cores included compression tests and measurements of density. In detail, one external brick and five cores taken from the internal layer were analysed. The brick showed a compression strength around 10 MPa and a Young's modulus of approximately 8000 MPa. On average, the results of the analyses of the cores reveal that the internal portion of the masonry ("sacco") is characterized by a compression strength of 4 MPa and a Young's modulus of 2500 MPa. It has to be noted that the basement seems to have higher mechanical properties (a compressive strength of 6 MPa and a Young's moduls of 4000 MPa). Density measurements provided the following values: 22-24 kN/m3 for the selenitic bricks at the base; 17-18 kN/m3 for the masonry bricks and 16-18 m3 for the internal fill. No measurements of tensile strengths and Poisson's coefficients are available for both the single materials (bricks and mortar) and the masonry assemblies. Moreover, no experimental data are available for the selenitic basement at all. Values obtained from other experimental tests (unpublished results of tests commissioned by the University of Bologna) on the selenitic basement of the near Garisenda tower (compressive strength equal to 11.9 MPa) may be considered as reasonable. The measured values as obtained from limited experimental tests have been validated/ compared with typical values from similar structures or values suggested by codes [6-12].

3 STRUCTURE MODELS

Models of increasing complexity are developed in order to evaluate the main dynamic properties of the Tower. All the models are based on linear elastic material behaviour. The assumption is consistent provided that the main objective is the assessment of the longitudinal and torsional natural periods of vibration of the Tower. The elastic properties collected in Table 3 are assumed to be representative of the average properties of the entire tower. First, the tower is idealized as a continuum model. Then, different (one dimensional and two dimensional) finite element models are developed.

3.1 Continuum models

The longitudinal and torsional vibrations of the tower are studied by schematizing the tower as a cantilever beam with fixed base conditions. As well known, the flexural and torsional free vibrations of a continuum beam are governed by the following differential equations (Den Hartog, 1947, [13]):

$$EJ(\mathrm{z})\frac{\partial^4 v}{\partial z^4} = \mu_1(\mathrm{z})\frac{\partial^2 v}{\partial t^2} \tag{1}$$

$$GJ_{\mathrm{t}}(\mathrm{z})\frac{\partial^4 v}{\partial z^2} = \mu_2(\mathrm{z})\frac{\partial^2 v}{\partial t^2} \tag{2}$$

where μ_1 is the mass per unit length; μ_2 is the moment of inertia per unit length ; $EJ(z)$ is the flexural stiffness of the beam (E is the Young's modulus, J is the moment of inertia); $GJt(z)$ is the torsional stiffness of the beam ($G=E/(2(1+\nu))$) is the shear modulus, J_t is the torsional moment of inertia). In the case of uniform beam, i.e. constant $EJ(z)$, $GJt(z)$, $\mu_1(z)$, $\mu_2(z)$, the fundamental flexural and torsional natural frequencies are given by:

$$\omega_{1,f} = \frac{4\pi^2}{l^2}\sqrt{\frac{EJ}{\mu_1}} \tag{3}$$

$$\omega_{1,t} = \frac{2\pi}{l}\sqrt{\frac{GJ_t}{\mu_2}} \tag{4}$$

where l is the height of the tower. If average values (along the height of the tower) are introduced in Eqs. 3 and 4, values of $T_{1,f} = 2\pi/\omega_{1,f} = 5.06$ and $T_{1,t} = 2\pi/\omega_{1,t} = 0.68$ are obtained.

In general, the ratio between the fundamental torsional and lateral periods (referred to as torsional-to-lateral frequency ratio, Ω_θ, see Chopra 1995 [14]) of a uniform (i.e. constant geometrical and mechanical properties along the height) tower can be estimated using the following relationship:

$$\Omega_\theta = \frac{\omega_{1,\mathrm{t}}}{\omega_{1,\mathrm{f}}} = 2\pi\frac{l}{i}\cdot\sqrt{\frac{J_t}{2(1+\nu)J}} \tag{5}$$

where $i = \sqrt{J/A}$ is the radius of inertia of the cross section of the tower. Typically, the i/l ratio varies between 0.01 (for slender towers) and 0.06 (for squat tower) leading to torsional-to-lateral frequency ratios between approximately 3.0 to 20.0 (assuming a Poisson's coefficient of 0.2). These values are common for torsionally-stiff structures.

As far as the actual tower cross section variation is concerned (to better represent the actual mass and inertia distribution), it is possible to obtain an approximate expression of the fundamental flexural frequency by using the Rayleigh's method [13]. The method is based on the principle of energy conservation and consists in assuming an approximate shape of the fundamental mode of vibration $\phi(z)$. By equating the maximum kinetic and potential energy the following expression of the approximate fundamental frequency can be obtained:

$$\omega_{1,f} = \sqrt{\frac{\int_l EJ(\mathrm{z})\phi''(\mathrm{z})^2 dz}{\int_l \mu(\mathrm{z})\phi(\mathrm{z})^2 dz}} \tag{6}$$

In the case of a slender tower, $\phi(z)$ can be assumed as the deformed shape (static analyses) of a uniform cantilever beam subjected to an inverted triangular distribution (the interested reader may find the derivation in the textbook of Belluzzi [15]), leading to:

$$\phi(\mathrm{z}) = A\left(\left(\frac{\mathrm{z}}{l}\right)^5 - 10\left(\frac{\mathrm{z}}{l}\right)^3 + 20\left(\frac{\mathrm{z}}{l}\right)^2\right) \tag{7}$$

Parameter A is an unknown scalar ($\phi(z)$ is an eigenvector).

By substituting Eq. 7 in Eq. 6, a fundamental period equal to 3.1 sec is obtained. It is worth to note that this value of period is significantly lower than the one obtained by assuming a uniform beam by averaging the cross section inertia and mass over the height of the tower. This result can be considered of general validity when dealing with slender masonry towers whose cross sections gradually decrease with the height.

3.2 Finite element models

In addition to the continuum models, finite element models of increasing complexity have been developed to evaluate the influence of the following specific issues to the dynamic properties of the tower: (i) soil-structure interaction, (ii) P-Δ effects and (iii) masonry orthotropy. The following six types of models have been developed:

- 1D-FB models: monodimensional (beam) models with fixed base conditions.
- 1D-SS models: monodimensional (beam) models including soil-structure interaction.
- 1D-SS-PD models: monodimensional (beam) models including soil-structure interaction and P-Delta effects.
- 2D-FB models: bidimensional (isotropic shell) models with fixed base conditions.
- 2D-SS models: bidimensional (isotropic shell) models including soil-structure interaction.
- 2D-O-SS-PD models: bidimensional (orthotropic shell) models including soil-structure interaction and P-Delta effects.

All the finite element models are developed using the commercial software SAP2000 [16]. The soil-structure interaction is modelled by using the simple method described in the book by Kramer 1996 [17] based on the approach of Wolf 1985 [18]. The soil is simulated by a system of equivalent linear springs and dampers (translational and rotational springs and dampers). The stiffness of the translational and rotational springs are obtained from the soil characteristics in terms of soil density, soil elastic properties (Young's modulus and Poisson's coefficient) and shear wave velocity. In the 2D-O-SS-PD model the orthotropic behavior of the masonry has been also accounted for by using different values of the Young's modulus along the vertical direction (E_v=4000MPa) and the horizontal direction (E_h=3000MPa). It should be noted that the monodimensional models do not allow to evaluate the torsional frequencies and moreover do not allow to evaluate the stresses.

The values of the first three flexural periods and the first torsional periods as obtained from the six finite element models are reported in Table 2. Inspection of the values reported in Table 6 allows the following observations:

- The fundamental periods of the two models with fixed based conditions (i.e. models MFB and models BFB) are between 3.37 s and 3.48 s. The values are close to analytical results as obtained with the Rayleigh's method (3.1 s)
- The inclusion of the soil-structure interaction causes an increase in the fundamental periods of approximately 10% and therefore should be accounted for an accurate evaluation of the fundamental periods. The effect of the soil-structure interaction on the other periods appear negligible;

- The other considered effects (P-Δ effects and the orthotropic behavior of the masonry) do not seem to have significant effects on the dynamic properties of the Tower.

Table 2: The properties of the soil surrounding the tower and the spring constants adopted in the finite element model.

Model name	Finite element	Base conditions	P-Δ	$T_{1,f}$ (s)	$T_{2,f}$ (s)	$T_{3,f}$ (s)	$T_{1,t}$ (s)
1D-FB	beam	fixed base	no	3.37	0.73	0.28	-
1D-SS	beam	soil-structure interaction	no	3.58	0.79	0.31	-
1D-SS-PD	beam	soil-structure interaction	yes	3.68	0.79	0.31	-
2D-FB	shell	fixed base	no	3.48	0.79	0.30	0.34
2D-SS	shell	soil-structure interaction	no	3.69	0.85	0.33	0.34
2D-O-SS-PD	shell	soil-structure interaction	yes	3.69	0.83	0.34	0.40

4 THE DYNAMIC EXPERIMENTAL TESTS BY THE INGV

From June to September 2012, measurements of the dynamic properties of the tower has been conducted by the INGV, following the recent 2012 Emilia Earthquake [19], which struck the northern regions of Italy causing extensive damages to masonry and industrial buildings and also cases of partial and global collapses (UCL report), by the INGV. Four triaxial Lennartz seismometers were installed inside the tower (one at the top, AS01, one at 70.0 m, AS02, one at 35.0 m, AS03 and one at the base AS04). A comprehensive description of the experiments is beyond the scope of the present paper. The interested reader may find all the detail in a report by the INGV [3].

Spectral analysis on the recorded data allows to identify the fundamental frequencies of the towers (Figure 2). The measured ranges of the first three lateral periods are in good agreements with the results of the numerical models.

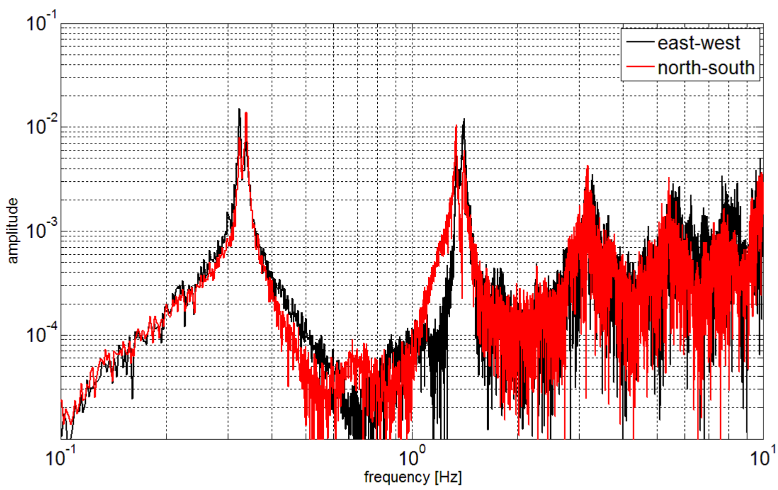

Figure 2. FFT of the recorded signal.

Figure 3 displays the free vibration response at the top of the tower in terms of displacement along the east-west and the north-south directions produced by a transient, propagating along the tower, that could be presumably associated to the transit of a bus. It can be clearly recognized the beating phenomenon which is typical of the free vibration response of one-storey eccentric systems ([20-22]). From a mathematical point of view, a complete beat is the sum of two sinusoidal waves of identical amplitude and close frequencies f_1 and f_2. The frequencies of the envelope response (or slow mode), f_{slow} and the fast mode f_{fast} are equal to $f_{slow} = (f_1 - f_2)/2$ and $f_{slow} = (f_1 + f_2)/2$, respectively. Partial beats, as those observed in Figure 3, may be the result of two harmonic components of slightly different amplitudes. The study and interpretation of this phenomenon is beyond the scope of the present paper and will be the object of future specific analyses.

The tower free vibration response also allows the evaluation of the damping ratio ξ by using the logarithmic decrement method [14]:

$$\xi = \frac{1}{2\pi n} \cdot \ln\left(\frac{U_1}{U_{n+1}}\right) \tag{8}$$

where U_1 and U_{n+1} are the amplitude of two consecutive peaks of the envelope response, while n is the number of fast waves within two consecutive peaks (i.e. a complete beat). For the Asinelli tower n varies between 50 and 54. In this case Eq. 8 leads to ξ values around 0.003-0.004. Such small damping ratios can be explained by the very small amplitudes of the recorded vibrations (10^{-4} m leading to drift ratios of 10^{-6}).

By assuming the measured signal as the sum of the responses of two SDOF damped system with equal damping ratio ξ, it is possible to estimate the frequencies of the two SODF f_1 and f_2 systems once the number of cycles n of a complete beat, the frequency f_{fast} and the ratios between the maximum and minimum amplitude in a complete slow cycle U_{max} and U_{min} (see Figure 4), are known.

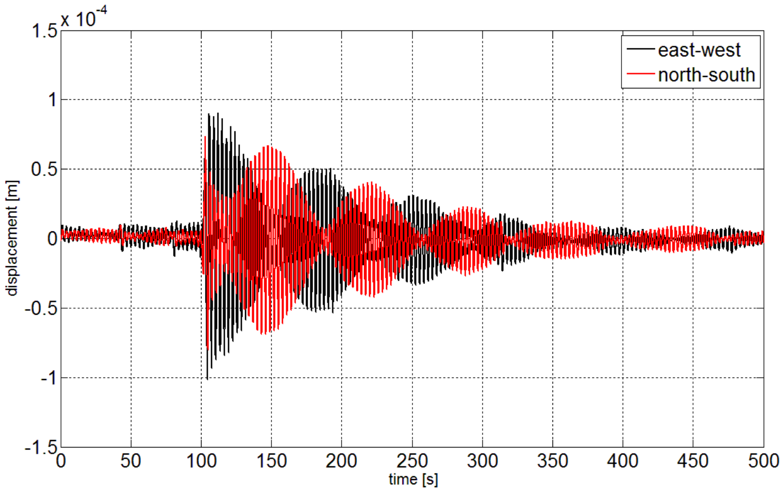

Figure 3. The free vibration response after the excitation due a signal that can presumably be associated to the transit of a bus.

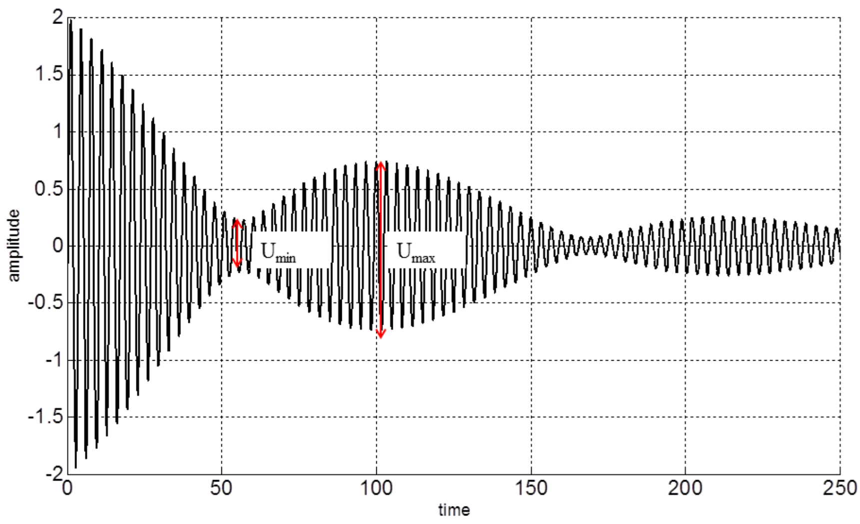

Figure 4. An uncompleted damped beat.

In the case of towers the fundamental frequencies of the SDOF systems can be interpreted as the fundamental (or first) flexural frequencies along the east-west and north-south directions (referred to as $f_{1,E\text{-}W}$ and $f_{1,N\text{-}S}$, respectively) and can be obtained by solving the following system of equations:

$$\begin{cases} f_{fast} = \dfrac{f_{1,N-S} + f_{1,E-W}}{2} \\ n = \dfrac{f_{fast}}{f_{slow}} = \dfrac{f_{1,N-S} + f_{1,E-W}}{f_{1,N-S} - f_{1,E-W}} \end{cases} \tag{9}$$

leading to:

$$\begin{aligned} f_{1,N-S} &= \frac{f_{fast}(n-1)}{n} \\ f_{1,\mathrm{E}-W} &= 2 f_{fast} - f_{1,N-S} \end{aligned} \tag{10}$$

Assuming n=48 and T_{fast}=3.05 s (from the measured response), the first flexural periods turn out to be equal to $T_{1,N\text{-}S}$=3.10 and $T_{1,W\text{-}E}$=2.97. In Figure 5 the simulated signal is superimposed to the measured one. A very good agreement can be observed.

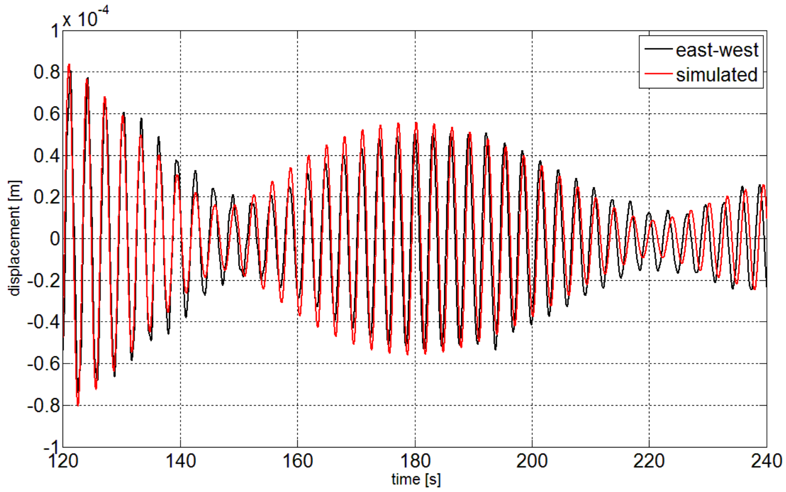

Figure 5. The comparison between the measured and the simulated response.

CONCLUSIONS

In the present paper, the mechanical and dynamic properties of the Asinelli Tower have been investigated. An approach to obtain a reliable mechanical characterization of the Tower in the presence of incomplete/insufficient experimental data has been proposed. It is based on the combined use of measured, assumed and calculated material properties.

First, the approach has been applied by using different material models. Then, elastic structure models (both simple continuum models and more complex finite element models) have been developed in order to evaluate the fundamental dynamic properties of the Tower (fundamental frequencies and damping ratio). The influence of specific aspects such as the actual mass and inertia distribution, the soil-structure interaction, P-Δ effects, and the masonry orthotropy on the dynamic properties of the Tower have been studied. Finally, the dynamic properties as obtained from the models are compared with the results of recent experiments carried out by the INGV.

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