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FE MODELLING FOR SEISMIC ASSESSMENT OF AN ANCIENT TOWER FROM AMBIENT VIBRATION SURVEY

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ABSTRACT

The paper exemplifies the application of the methodology involving historical and architectural research complemented by ambient vibration survey in the structural assessment of the so-called Zuccaro's tower in Mantua, Italy. The tower is about 43.0 m high and built in solid brick masonry. The good knowledge of the structural geometry and the large number of identified vibration modes allowed to establish a 3D numerical model of the tower for the quantitative assessment of its seismic vulnerability. It is worth underlining that: (a) the elastic parameters of the model, which were identified by minimizing the difference between numerical and experimental resonant frequencies, exhibited an excellent agreement with the available characterization of the masonry material (which was found in the archives and dates back to the '90s); (b) the proposed methodology involves only non-destructive tests and might allow relatively quick seismic assessment of ancient towers at the territorial level.

Keywords: Non-destructive testing, Masonry towers, Ambient vibration test, Model updating

1. INTRODUCTION

The assessment of ancient masonry towers is a challenging multidisciplinary activity, involving different tasks. Once the geometry of the building has been retrieved through the topographic survey, qualitative information results from historical research and direct inspections so that important data are made available on the geometric arrangement, the construction techniques and the mapping of discontinuities and damages. Operational modal testing and analysis (even carried out using a reduced number of sensors) could supplement the previous investigation steps, contributing to the diagnosis of historic towers with quantitative parameters, which are representative of the structural condition.

The paper exemplifies the application of the methodology involving historical and architectural research, complemented by the dynamic survey, for the structural assessment of the Zuccaro's tower in Mantua, Italy (Fig. 1). The tower is approximately 43 m high, built in solid brick masonry with eight timber floors and a masonry cross-vault covering the ground level. The relatively good knowledge of the structural geometry, along with the complete inspection carried out and the large number of identified vibration modes allowed to establish a 3D numerical model of the tower. It is worth underlining that:

(a) the developed model reproduces with good accuracy the geometry and out-of-plumb of the tower, the connection with the neighboring buildings, the stiffening of timber floors and the observed characteristics of the lower 7 vibration modes;

(b) the elastic parameters of the model, which were identified by minimizing the difference between numerical and experimental resonant frequencies, exhibited an excellent agreement with the available characterization of the masonry material (which was found in the archives and dated back to the '90s). Furthermore, the proposed methodology involves only non-destructive tests and might allow relatively quick seismic assessment of ancient towers at the territorial level.

2. THE ZUCCARO'S TOWER IN MANTUA, ITALY

The Zuccaro's Tower (Fig. 1) is an ancient defensive structure built in Mantua during the Middle Ages. It is about 43 m high and has an approximately square plan with a side of 8.5 m. The masonry walls are built in solid bricks and lime mortar with a thickness ranging from 1.1m, at the base, to 0.8m at the top. The tower includes eight timber floors and a masonry cross vault at the first level. Furthermore, it is surrounded by buildings on three sides up to the height of 10 m, and the only entrance is from the S-W side, toward *don Tazzoli* Street.

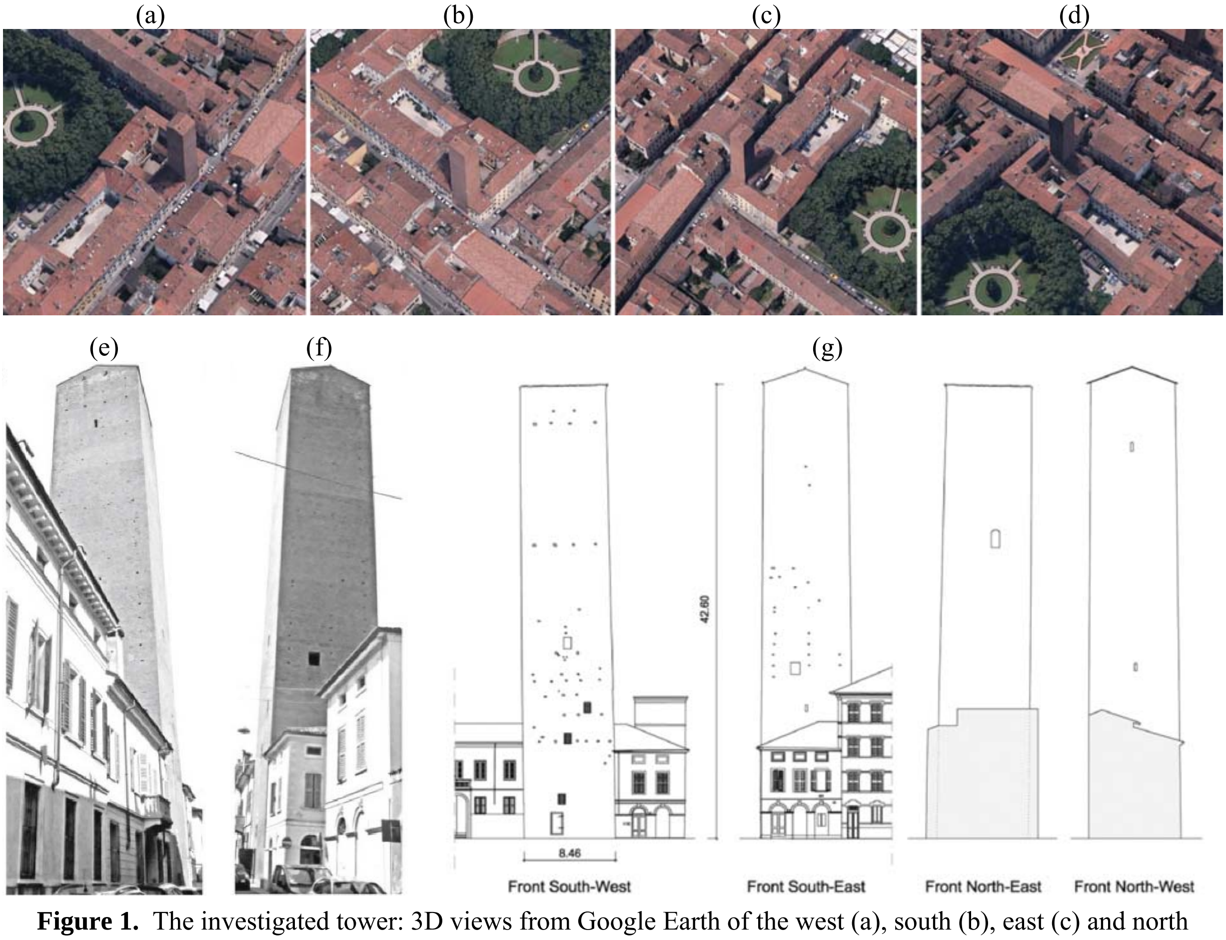
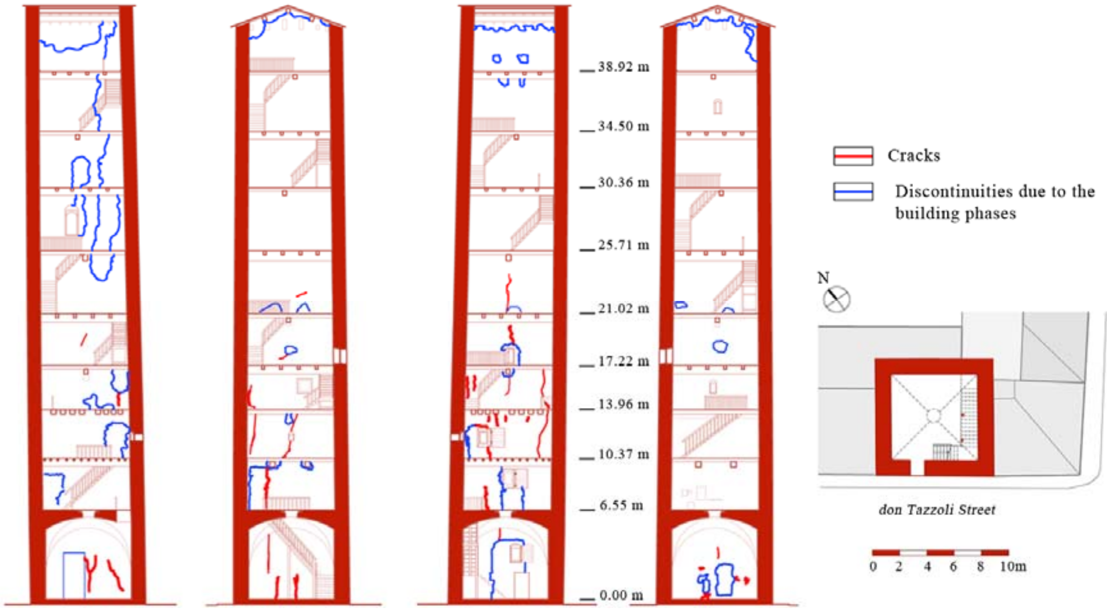


Figure 1. The investigated tower: 3D views from Google Earth of the west (a), south (b), east (c) and north corners (d); pictures from the ground level from the west (e) and south corners (f); and geometric survey of the external façades (g).

(a) (b) (c)

Figure 2. Details of masonry cracks and discontinuities: (a) ground floor, N-E front, (b) 1st floor, S-E front and (c) 3rd floor, S-W front.



N-E front S-E front S-W front N-W front

Figure 3. Damage survey and crack patterns of the inner fronts.

2.1. Historical and documentary research

The original defensive role of the structure is demonstrated by its morphology and position: the openings are very few and small, and the location was at the limits of the Middle Ages fortifications. As reported by Saisi et al. [1], the first historical record regarding the existence of the tower dates back to 1143, but the lack of information regarding several transformations makes difficult to reconstruct the evolution phases of the structure completely. In 1533 the building was subjected to some not well-documented repairing interventions after the damages caused by lightning, while in 1717 a new access from *don Tazzoli* Street was opened and the roof rebuilt. Other information regarding the relationship with the neighboring constructions is obtained from the historical cartography [1]. During the XVI and XVII centuries, the tower was represented in a building aggregate, surrounded by low-rise buildings on two sides, with an entrance from the N-E front. Nowadays, the only entrance is on the opposite side, the former one was closed (Fig. 2a), and another construction is present on N-E side. Another map from the first half of the XIX century reveals the presence of an external staircase, which probably connected *don Tazzoli* Street with an upper entrance, as confirmed by the presence of infilled openings on that front (Fig. 2b).

More recently, in the 60s, some documents from the Local Superintendence reported the concerning about the bad state of preservation of the tower, requesting necessary inspections and interventions. Subsequently, a fire occurred in 1979, and several interventions were carried out in 1982, from the substitution of timber floors, roof and internal staircase to local masonry rebuilding, and the opening of new windows. Between 1994 and 1997 an extensive experimental campaign was carried out by ISMES [2], including the installation of a static monitoring system and the characterization of masonry materials: the average elastic modulus identified from the double flat jack tests (DFJT) at the base level was equal to 3.18 GPa. After the campaign, a strengthening intervention was carried out involving the substitution of the roof and timber floors, and the injection of expanding mortars in the main cracks.

2.2. Recent on-site investigations

An extensive campaign devoted to geometry and crack surveying was recently carried out by Saisi et al. [1]. Through visual inspection, it was possible to identify some sharp discontinuities and deep cracks around the corner between the S-W and S-E inner fronts (Fig. 2c) starting from the base up to 17.22 m. Furthermore, it was possible identifying some deep and thick cracks on the cross vault at the first level and extended dark areas on the inner walls' surfaces probably caused by the fire occurred in 1979. Furthermore, it is worth considering the presence of large areas with fragmentary and non-homogeneous brick masonry, especially in the N-E inner front, starting around 21.0 m (Fig. 3).

3. AMBIENT VIBRATION TESTS AND OPERATIONAL MODAL ANALYSIS

3.1. Testing procedure

Two ambient vibration tests were carried out to study the dynamic behavior of the tower. The first test was performed on 23-24 October 2016 with few sensors (i.e., four seismometers) placed at the top floor (Fig. 4a). A second test was carried out on 11-12 December 2017 with a larger number of measuring points (Fig. 4b-c) to evaluate the mode shapes of the building.

During the test of October 2016, two opposite corners at the 9th floor were instrumented employing four seismometers (electro-dynamic velocity transducers, SARA SS45, Fig. 4d). The structural response induced by ambient vibrations was recorded for twelve hours, during the night, with temperatures ranging between 10.3°C and 12.6°C, and a sampling rate of 200 Hz. The data were stored in 12 sets of 3600 s.

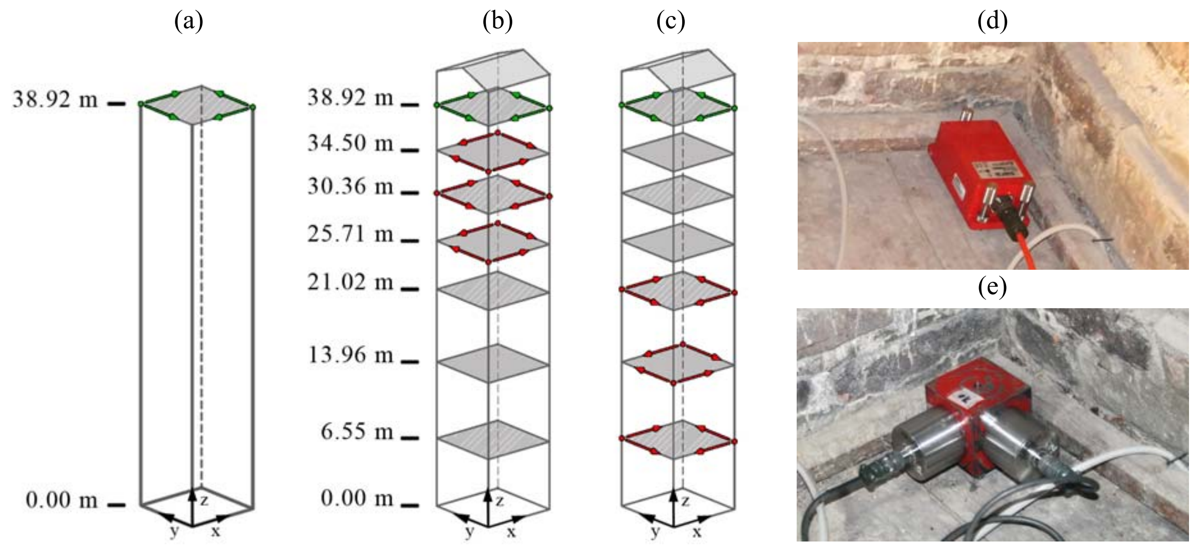


Figure 4. Sensors layout in the AVTs of the 23-24 October 2016 (a) and 11-12 December 2017 (b)-(c), and sensors employed in the first (d) and second (e) test.

(a) (b)

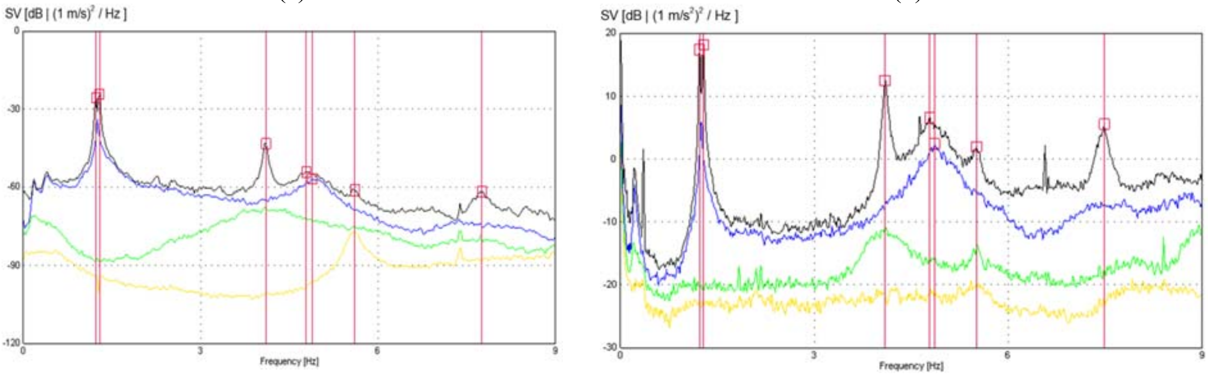


Figure 5. Singular value lines (FDD) and identification of natural frequencies from the data collected in October 2016 (a) and December 2017 (b).

The test of December 2017 was conducted with 14 points instrumented along the height of the tower, employing 28 WR 731A accelerometers (10 V/g sensitivity and ± 0.50 g peak acceleration, Fig. 4e) and a multi-channel acquisition system with NI 9234 data acquisition modules (24-bit resolution, 102 dB dynamic range and anti-aliasing filters). During the test of December 2017, the temperature ranged between −0.1°C and +2.0°C and the sampling frequency was set equal to 200 Hz.

3.2. Operational modal analysis (OMA) and results

As previously pointed out, the modal identification was performed by considering the accelerations induced by micro-tremors and wind. Time series of 3000 s were considered, and different output-only identification algorithms were employed to obtain an estimate of natural frequencies and mode shapes. The Frequency Domain Decomposition (FDD) [3] and the data-driven Stochastic Subspace Identification (SSI-data) [4], both available in the commercial software ARTeMIS [5], were applied to the recorded data.

The local maxima of the first singular value (SV) line, obtained from the processing of the recorded data, represent the vibration modes of the structure. Fig 5 shows the first four SV lines obtained from both tests where 7 modes were identified in the frequency range of 0-9 Hz. The estimates of the mode shapes are shown in Figs. 6 and 7 for the tests of October 2016 and December 2017 respectively. The sequence of the first five modes is consistent with past experimental studies on similar towers [6, 7, 8], while the last two modes seem to be very peculiar of this structure. In more detail, the first two modes are closely spaced well-defined bending modes in opposite directions (B_{y1} and B_{x1}), followed by a torsion mode (T_1) and another couple of higher-order bending modes (B_{y2} and B_{x2}). The peculiar shape of the last two modes is known as warping (W_1 and W_2) [1], which represent a distortion of the structure's cross-section (Figs. 7f-g). In the case of the investigated tower, the warping distortion is conceivably related to the length of the structure's side and the absence of rigid floors.

Regardless the differences in the external temperatures between the two tests, the excellent correspondence of the first five frequencies (Tab. 1) demonstrates that the global dynamic behavior of the tower can be evaluated by sensing just the upper level, confirming the possibility of permanently installing in the structure a cost-effective monitoring system employing 3-4 accelerometers or seismometers. It is further noticed that the higher modes (W_1 and W_2) seem to exhibit a stronger sensitivity to the variations of temperatures.

4. FINITE ELEMENT MODELLING AND MODEL UPDATING

4.1. The Abaqus model

The numerical simulations were carried out with the FE code ABAQUS [9] by means of a detailed 3D model using eighth-node brick elements (C3D8). To obtain a regular distribution of masses and a good representation of the opening distribution and thickness variations, a relatively large number of

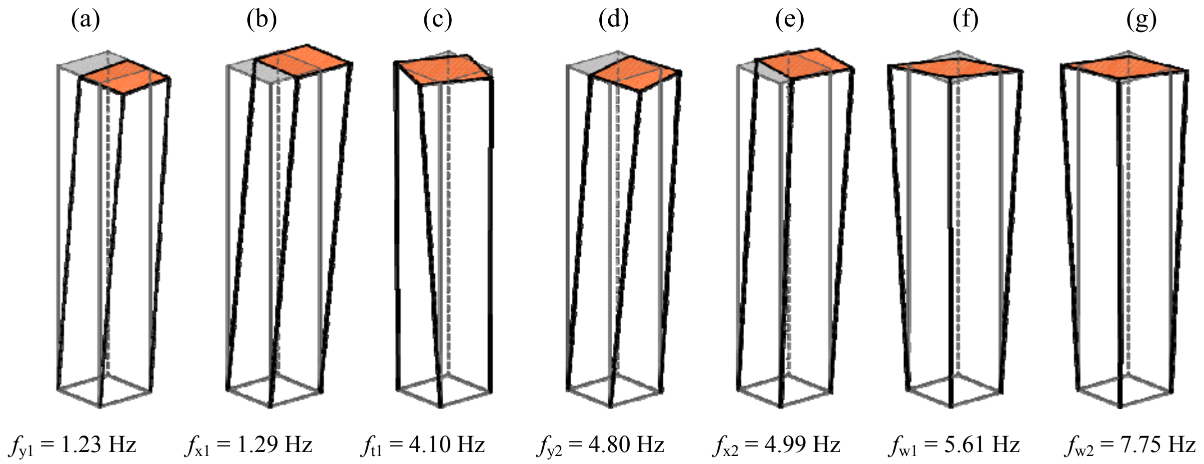


Figure 6. Vibration modes identified employing few seismometers (October 2016).

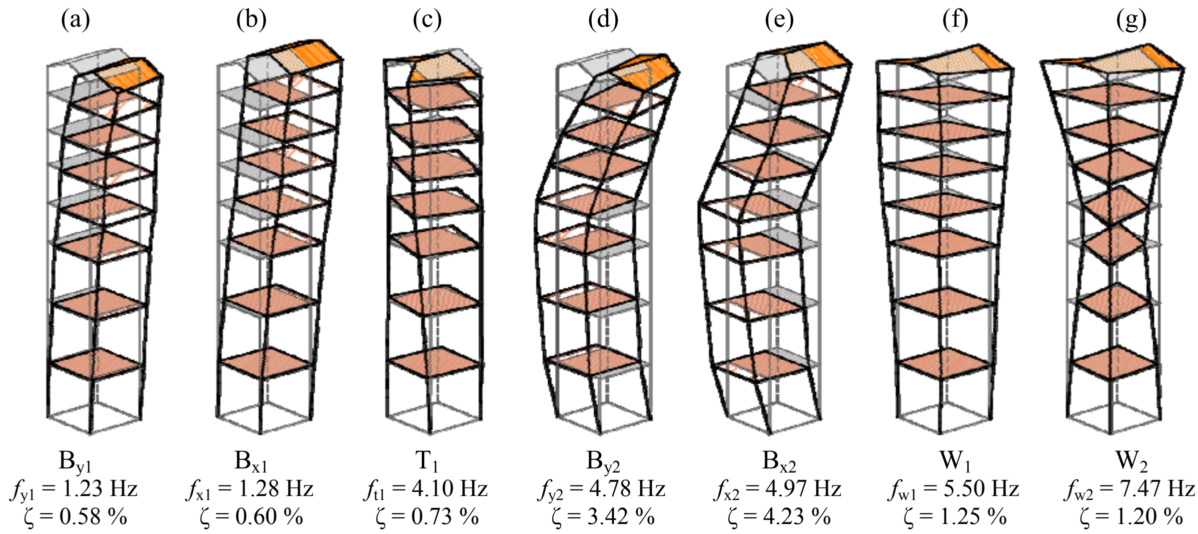


Figure 7. Vibration modes identified employing a large number of accelerometers (December 2017).

Table 1. Comparison between the natural frequencies identified in October 2016 and December 2017.

Mode no.	Mode type	$f_{oct\text{-}2016}$ (Hz)	$f_{dec\text{-}2017}$ (Hz)	Error (%)
B_{y1}	Bending mode in N-E/S-W direction	1.23	1.23	0.00
B_{x1}	Bending mode in N-W/S-E direction	1.29	1.28	0.78
T_1	Torsion mode	4.10	4.09	0.24
B_{y2}	Bending mode in N-E/S-W direction	4.80	4.78	0.42
B_{x2}	Bending mode in N-W/S-E direction	4.99	4.97	0.40
W_1	Warping distortion mode	5.61	5.50	1.96
W_2	Warping distortion mode	7.75	7.47	3.61

elements have been used: the average element size is approximately equal to 0.5m. In addition, the mesh size is sufficiently refined to provide a negligible effect on modal parameters. Overall, the FE model consists of 10582 brick elements with 48438 active degrees of freedom (Fig. 8).

The vibration-based FE model updating (FEMU) of historic towers is mainly connected with the calibration of masonry elastic properties (E and G) and boundary conditions as reported by different scholars [6, 10]. Consequently, to reduce the number of updating parameters and consider just the ones affected by major uncertainties, some assumptions were introduced: (a) the effect of soil-structure interaction was neglected, and the tower was considered fixed at the base; (b) regarding the modelling of brick masonry, a linear elastic orthotropic material was adopted; (c) the relationship between Young's modulus and the shear modulus was considered equal to $G = \alpha \cdot E$; (d) a homogeneous distribution of weight per unit volume was assumed equal to 17 kN/m^3; and (d) a Poisson's ratio of 0.15 was held constant.

(a) (b) (c) (d) (e) (f)

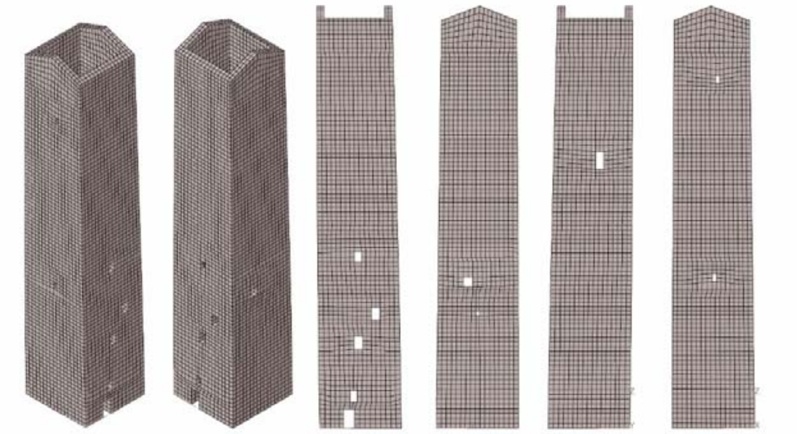


Figure 8. FE model of the Zuccaro's Tower, axonometric views of the West (a) and South corner (b), and parallel projections of the S-W (c), S-E (d), N-E (e), and N-W (f) façades.

(a) (b) (c) (d)

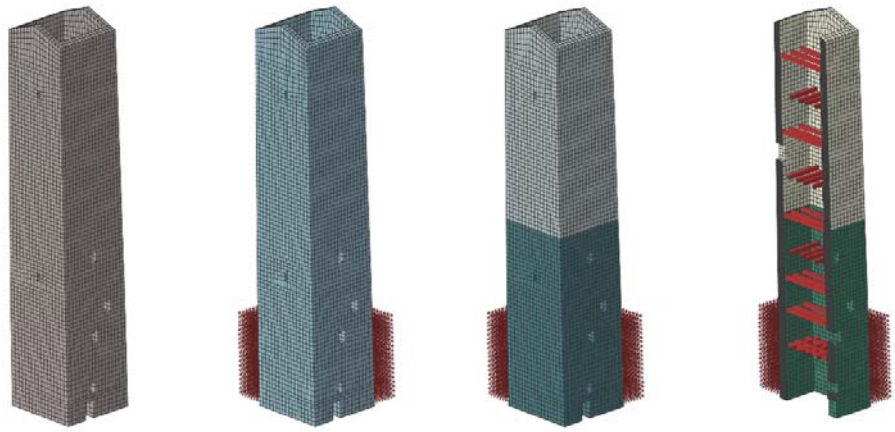


Figure 9. Developed models: (a) Model 0, (b) Model 1, (c) Model 2, and (d) Model 3 (optimised model).

To demonstrate the need for FEMU procedures, an initial model (Model 0, Fig. 9a) was developed just considering the results of the flat jack tests from the experimental campaign of ISMES [2] and the recommendations of the Italian Technical Code, without taking into consideration the effect of the nearby constructions. Thus, a FEMU technique based on the Douglas and Reid method [11] and the Particle Swarm Optimisation (PSO) algorithm [12] was applied to improve the correlation with experimental results. The response features chosen for the updating procedure were the natural frequencies. Despite the accurate geometry of the model and the quality of the structural identification procedures, some further considerations were needed to accurately represent all identified modes:

(1) the effect of surrounding buildings was simulated with a uniform distribution of 342 linear elastic translational springs on the S-E side and N-E side, with constants k_x and k_y respectively (Model 1, Fig. 9b);

(2) the presence of discontinuities and fragmentary masonry areas was simulated adopting a non-homogeneous distribution of Young's modulus E, namely, dividing the structure into two parts with constant elastic properties (Model 2, Fig. 9c);

(3) at last, the stiffening effect of the timber floors was simulated through a series of rigid beam elements connected with the vertical walls, thorough linear rotational springs with constant k_{RS} (Model 3, Fig. 9d).

Table 2. Summary of the developed FE models and corresponding average frequency discrepancy (DF_{ave}).

	Model 0	Model 1	Model 2	Model 3
Parameters optimization		×	×	×
Surrounding buildings effect		×	×	×
Effect of non-homogeneous masonry			×	×
Effect of timber floors				×
Number of vibration modes with DF < 1 %	0	3	5	7
DF_{ave} [%]	9.96	5.34	2.60	0.07
DF_{max} [%]	21.5	15.6	10.1	0.2

Hence, applying the sequence of the above steps (involving different assumptions on the modes' uncertainties), it was possible to obtain an excellent correlation with all identified natural frequencies.

4.2. Comparison between analytical and experimental results

The initial model (Model 0, Fig. 9a) was developed considering just the results of the material characterization campaign – average Young's modulus identified equal to 3.2 GPa – and the recommendations of the Italian Technical Code – $G = E/3$ –, without considering the effect of the nearby constructions. The correlation between the numerical and experimental response is measured through the frequency discrepancy, defined as follow:

$$DF_i = (f_i^{EXP} - f_i^{FEM}) / f_i^{EXP}\ 100 \quad [\%] \tag{1}$$

and particularly with its average DF_{ave} and the maximum DF_{max} index. Therefore, the initial model (Model 0, Fig. 9a) exhibits a poor correlation with the OMA results, showing a much stiffer response with all the natural frequencies exceeding the experimental ones, and a DF_{ave} and DF_{max} equal to 10% and 22% respectively (Tab. 2). Furthermore, it is worth to mention that Model 0 would have been conceivably very similar to a standard model developed according to the Italian Code, without the information collected with AVTs.

To improve the correlation with experimental results, Model 1 (Fig. 9b) considers the effect of the building aggregate where the tower is enclosed through a series of linear elastic springs. Based on the sensitivity analysis, the parameters selected for the model updating procedure were the average masonry elastic modulus E, the ratio $\alpha = G/E$, and the resulting spring stiffness in the two main directions $\sum k_x$ and $\sum k_y$. Hence, the optimal parameters provided by the model updating algorithm are reported in Table 3. As expected, there is a global reduction of the masonry elastic properties: the response of Model 0 was indeed much stiffer than the (real) structural response. Furthermore, the springs simulating the interaction with the neighboring buildings in the two directions have very different stiffness, presumably for the different morphology of surrounding constructions. In y-direction, the constraint effect is given by an entire building aggregate while in x-direction just an isolated block is present (Fig. 1a). Although the overall correlation was improved, and the first three resonant frequencies (B_{y1}, B_{x1}, and T_1) were well reproduced (Table 4), the numerical model is still stiffer than the real structure with four modes out of seven exceeding the experimental ones, apparently due to the presence of discontinuities in the masonry walls.

Model 2 (Fig. 9c) was implemented assuming two regions with constant elastic properties: E_L for the lower part (height ≤ 21.02 m) and E_U for the upper part (height > 21.02 m). The division of the areas at the height of 21 m (the 5th floor) is consistent with the starting point of non-homogeneities in the masonry of the N-E front, reported by the direct survey. As expected, the parameters resulting from the calibration show a remarkable reduction of the elastic properties of the upper part (E_U) while in lower part just a slight increase is reported (Table 3). Model 2 is capable of reproducing with high accuracy the lower 5 modes of the real structure (with the maximum frequency discrepancy being 0.4%), whereas it turns out to be too flexible to correctly reproduce the warping modes; this relative lack of stiffness is most likely due to the neglected effect of timber floors.

Finally, to improve the model calibration and the identification of the warping modes, it was necessary to consider the effect of the timber floors (Model 3, Fig. 9d). It is worth noting that these floors were substituted in the ‘90s, and the quality of the connections between the timber elements and the masonry walls is reasonably not negligible. Subsequently, rigid elements were placed at each floor, parallel to the rigid direction of the timber floors, to simulate the connections between the opposed masonry walls. Each connection between a rigid element and a brick element was modeled with a linear elastic rotation spring of constant k_{RS}. The model updating procedure led to the identification of the structural parameters reported in Table 3. The stiffening effect related to the introduction of rigid elements is clearly reflected in a slight decrease of both Young’s moduli and compensated by an increase of ratio α, in a way that leaves the shear moduli approximately constant. Interestingly, the constants of both springs increase, demonstrating the difficulty of a clear identification most likely connected with the low sensitivity of these parameters. On the other hand, the numerical frequencies obtained with the above set of parameters are basically equivalent to the one estimated from the AVT of December 2017 with a DF_{max} much less than 1% (Fig. 10).

Table 3. Summary of the identified structural parameters.

Structural parameters	Model 0	Model 1	Model 2	Model 3
E (GPa) whole structure	3.20	2.97	-	-
E_L (GPa) height ≤ 21.02m	-	-	3.14	3.07
E_U (GPa) height > 21.02m	-	-	1.71	1.56
A	0.333	0.298	0.333	0.342
G (GPa) whole structure	1.067	0.885	-	-
G_L (GPa) height ≤ 21.02m	-	-	1.046	1.052
G_U (GPa) height > 21.02m	-	-	0.570	0.535
Σk_x (kN/m $\cdot 10^5$)	-	11.80	15.05	22.33
Σk_y (kN/m $\cdot 10^5$)	-	0.20	0.20	3.50
k_{RS} (kN/m $\cdot 10^5$)	-	-	-	0.169

Table 4. Comparison between experimental and numerical modal frequencies.

	f (Hz)				
Mode no.	FDD (2017)	Model 0	Model 1	Model 2	Model 3
B_{y1}	1.23	1.285	1.230	1.230	1.228
B_{x1}	1.28	1.295	1.281	1.281	1.280
T_1	4.10	4.433	4.096	4.100	4.100
B_{y2}	4.78	5.502	5.18	4.800	4.779
B_{x2}	4.97	5.436	5.37	4.974	4.969
W_1	5.50	6.680	6.36	4.94	5.499
W_2	7.47	8.224	7.87	6.91	7.486

5. CONCLUSIONS

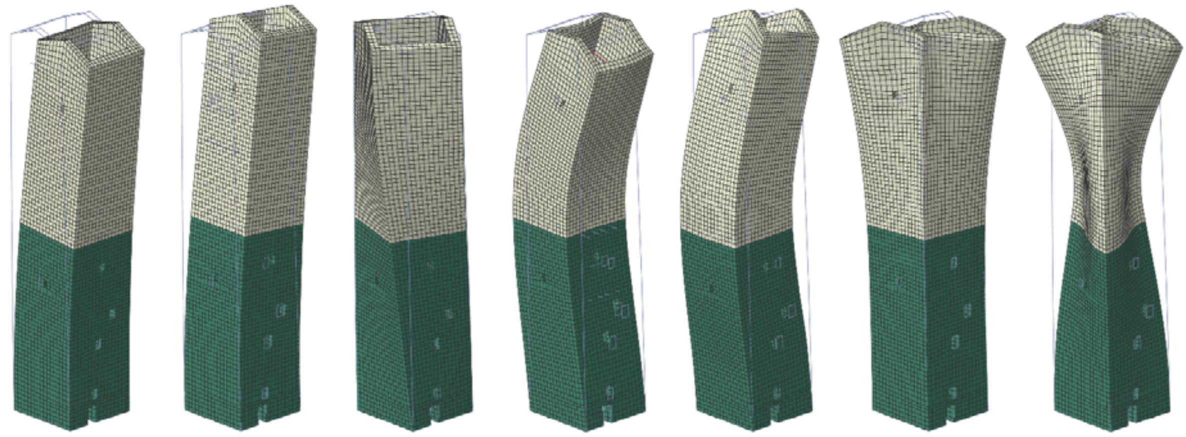
The paper focuses on the structural identification of the Zuccaro’s Tower in Mantua. After the seismic events of May 2012, a preliminary structural assessment of the state of preservation of the tower was performed [1]. During these investigations, the presence of non-homogeneous and fragmentary masonry areas was documented and confirmed by ambient vibration tests reporting anomalous distortions of the tower’s cross-sections. To better understand the effects of these non-homogeneities and anomalies, deeper analyses were suggested. Therefore, the current research programme involved two main steps: (a) the development of an FE model that represents accurately the information collected on-site, involving structure’s geometry, masonry quality and boundary conditions; and (b) the identification of uncertain model parameters by means of a FEMU technique based on the Douglas-Reid method and the PSO algorithm.

Based on the experimental data collected during an ambient vibration test, the FE model is iteratively calibrated. The results of the entire process summarised in Tables 2-4, suggest the following conclusions:

(1) Notwithstanding the very low level of excitation during the AVTs, seven vibration modes were identified in the frequency range of 0-9 Hz;

(2) Besides the initial model (Model 0) represented accurately the geometry surveyed and the material properties identified with the material characterization campaign, a very poor correlation with the actual structural response was obtained (DF_{ave} = 10%);

(3) On the contrary, applying the developed FE model updating procedure and considering the effects of local masonry non-homogeneities and timber floors, an excellent correlation with the experimental results was obtained (DF_{ave} = 0.05%).



(a)	(b)	(c)	(d)	(e)	(f)	(g)
B_{y1}	B_{x1}	T_1	B_{y2}	B_{x2}	W_1	W_2
f_{EXP} = 1.23 Hz	f_{EXP} = 1.28 Hz	f_{EXP} = 4.10 Hz	f_{EXP} = 4.78 Hz	f_{EXP} = 4.97 Hz	f_{EXP} = 5.50 Hz	f_{EXP} = 7.47 Hz
f_{FEM} = 1.228 Hz	f_{FEM} = 1.280 Hz	f_{FEM} = 4.010 Hz	f_{FEM} = 4.779 Hz	f_{FEM} = 4.969 Hz	f_{FEM} = 5.499 Hz	f_{FEM} = 7.486 Hz
DF_{y1} = 0.17 %	DF_{x1} = 0.02 %	DF_{T1} = 0.00 %	DF_{y2} = 0.01 %	DF_{x2} = 0.02 %	DF_{W1} = 0.03 %	DF_{W2} = -0.21 %

Figure 10. Vibration modes of the updated Model 3 and comparison with the experimental results.

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