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FE Modeling of a Historic Masonry Tower and Vibration-based Systematic Model Tuning

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Abstract The paper presents and discusses the procedure developed for the calibration of the structural FE model of a Bell-Tower, within a wide investigation program aimed to support the preservation and restoration actions. Ambient vibrations induced by wind, micro-tremors and swinging of bells were recorded and the identified modal parameters, together with geometric and crack pattern survey were used to calibrate a 3D F.E. of the tower.

Keywords: Dynamic test, masonry, model tuning

Introduction

The international debate concerning the preservation of Architectural Heritage states that a correct approach is based on the diagnosis of the historic buildings (Binda et al. 2000). This strategy involves the application of non destructive tests, according to the building problems. In the structural assessment, investigations on history, geometry, materials and damage provide a first diagnosis, to be further refined by using Finite Element (FE) analyses. The FE model of an historic structure, even when based on accurate field survey, always involves simplifying assumptions and several uncertainties in the material, the geometric properties and boundary conditions. Within this context, one possible key role of ambient vibration testing (AVT) and operational modal analysis (OMA) is to provide an effective and accurate validation of the model prior to its use in numerical analysis (see e.g. Gentile and Saisi 2007).

The paper details the main step of the model calibration of an historic bell tower in stonework masonry. The developed procedure of vibration-based model tuning, is based on the availability of experimental information related to: (a) accurate survey of the tower geometry and of the crack pattern; (b) AVT and identification of the dynamic characteristics of the tower (i.e. natural frequencies and mode shapes). linear elastic improved model, accurately fitting the modal parameters of the tower in its present condition. Subsequently, the proposed numerical procedure involves the following steps: (1) prior identification of the uncertain parameters of the model; (2) sensitivity analysis, aimed at evaluating the minimum number of parameters, which are good candidates for the model tuning; (3) systematic manual tuning aimed at estimating a model exhibiting an acceptable correlation with the experimental modal parameters (i.e. one-to-one correspondence with experimental mode shapes and limited discrepancies with respect to the observed resonant frequencies); (4) evaluation of the optimal values of the updating parameters by using a simple system identification technique (Douglas and Reid 1982) and verification (if possible) of the optimal estimates by physical evidence; (5) possible further improvement of the model by selecting an increased number of parameters and repeating steps (c)-(d).

The Case History: the Bell Tower of San Vitttore Church in Arcisate

The investigated bell tower (Fig. 1), about 37.0 m high, is built in stonework masonry and connected, on the East side and partly on the South side, to the XIth century church of San Vittore in Arcisate (Varese). The first historic document on the tower dates back to XVIth century, even probably built on a previous roman building and modified along the centuries. Six orders of floors are present; the last two orders were probably added in the 18th century to host the bell trusses. Along all sides, the tower exhibits long vertical cracks, most of them cutting the entire wall thickness and passing through the keystones of the arch window openings. These cracks are mainly distributed between the second / third order of the tower. Many superficial cracks are also diffused, particularly on the North and West fronts (Fig. 1), which are not adjacent to the church. The stonework masonry texture appears disordered with diffuse continuous or little staggered vertical joints and erosion of the mortar joints; it is, then, difficult to distinguish between insufficient stone interlocking and vertical cracks.

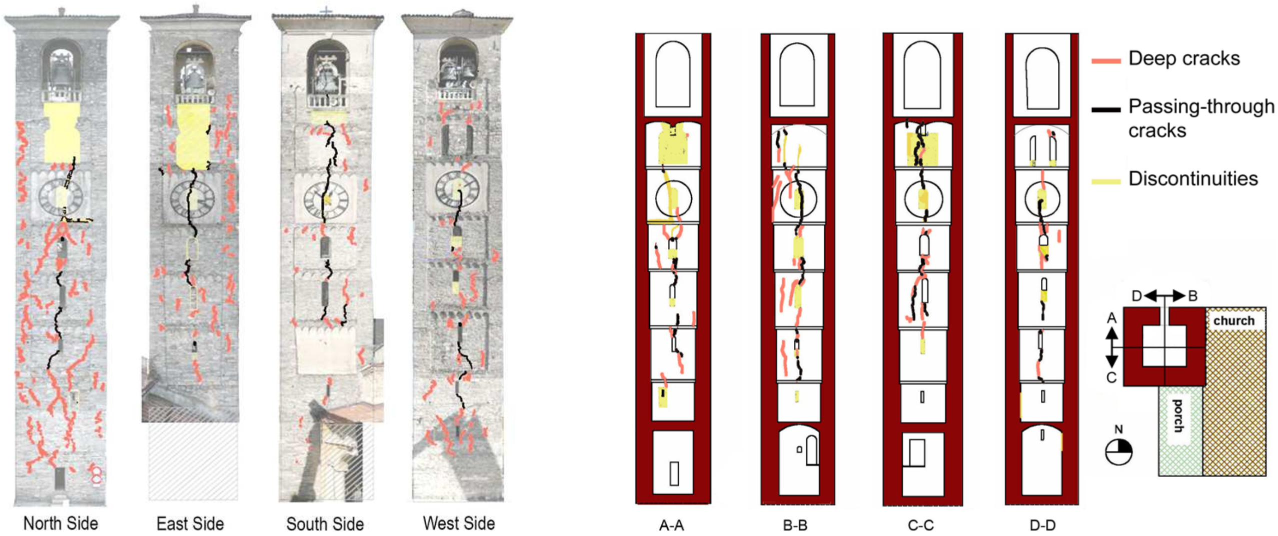


Figure 1: Crack pattern on the different fronts of the tower and on sections

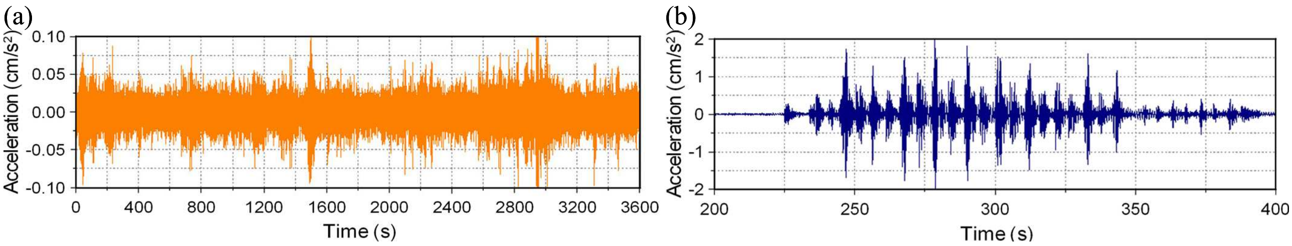


Figure 2: Sample of recorded accelerations under: (a) micro-tremors and wind; (b) swinging of bells

Testing Procedure and Data Analysis

Ambient Vibration Tests Two ambient vibration tests were conducted on the tower, on June 2007 and June 2008. In both tests, a 16-channel data acquisition system with 15 uniaxial WR 731A piezoelectric accelerometers were used. For each test, two different series of ambient vibration data were recorded: in the first series, referred in the following as AV1, the ambient excitation was only provided by the wind and the micro-tremors; in the second series, referred in the following as AV2, the excitation was provided by the swinging of bells. In both series, the well-known rule of thumb (see e.g. Cantieni 2005) about the length of the time windows acquired (that should be at least 1000 times the period of the structure's fundamental mode) was largely satisfied. The sample rate was 200 Hz to provide good waveform definition. An example of the acceleration time-histories recorded in June 2007 test in the upper part of the tower is given in Figs. 2(a)-(b). An important remark concerns the significant increase of the vibration level associated to ambient excitations compared to micro-tremors, as it is shown in Figs. 2(a)-(b); the maximum amplitude of acceleration responses is increased of about 20 times by the bell swinging. Similar results have been obtained in the test performed on June 2008.

Mode Identification The extraction of modal parameters from ambient vibration data was carried out by using two different output-only techniques: the Frequency Domain Decomposition (FDD, Brincker et al. 2000) in the frequency domain and the data driven Stochastic Subspace Identification (SSI, van Overschee and De Moor 1996) in the time domain; these techniques are available in the commercial program ARTeMIS (SVS 2010). In order to compare the mode shapes identified using different methods and different test data, the Modal Assurance Criterion (MAC, Allemang and Brown 1983) was computed.

Experimental Evidence 5 vibration modes were identified in the frequency range of 0-6 Hz by FDD and SSI techniques applied to AV1 data series. The results of OMA in terms of natural frequencies can be summarized through the plots of Figs. 3(a)-(b); the figures show the lower Singular Values (SV) of the spectral matrix (FDD, Fig. 3(a)) and the stabilization diagrams (SSI, Fig. 3(b)), respectively. Fig. 3(a) highlights the effectiveness of FDD technique in the mode identification through well-defined local maxima in the 1st SV line; similarly, Fig. 3(b) shows the alignments of the stable poles in the stabilization diagram of the SSI method, identifying the vibration modes, as well. Furthermore, Figs. 3(a)-(b) clearly show the correspondence of the natural frequency estimates between the two techniques, with the resonant peaks of Fig. 3(a) being placed at the same frequencies of the alignments of stable poles of Fig. 3(b).

Fig. 3(c) shows the identified mode shapes (June 2008, FDD technique): dominant bending (B) modes were identified at 1.21 (B_1), 1.29 (B_2), 3.98 (B_3) and 4.14 Hz (B_4) while only one torsion mode (T_1) was identified at 3.56 Hz. It is observed that the dominant bending modes of the tower involve flexure along the diagonals of the tower.

Further remarks derive from the comparison of the modal parameters identified from the two different data-sets acquired, under different level of ambient excitation, on June 2007, (Table 1): the comparison reveals slight but systematic decreases of the frequencies associated to the higher level of excitation caused by the bell swinging (Table 1). Furthermore, significant differences are detected between the mode shapes identified from data series AV1 and AV2. Similar results were obtained by applying the SSI technique and in the test performed on June 2008, even if with less marked differences in the frequency shift. The comparison of the mode shapes identified in the two series of tests clearly highlights that the MAC in the 2007 test tends to decrease as the order of mode increases; for the two upper modes the MAC is between 0.87 and 0.80. In the June 2008 test, the MAC seems more stable.

Hence, the dynamic characteristics of the tower are possibly dependent on the amplitude of excitation/response. In order to better understand and explore this peculiar behaviour a continuing dynamic monitoring has been installed.

Table 1: Correlation between the modal parameters identified in 2007 and 2008

	June 2007				*June 2008*			
Mode Type	f_{AV1} (Hz)	f_{AV2} (Hz)	$\Delta f/f$ (%)	*MAC*	f_{AV1} (Hz)	f_{AV2} (Hz)	$\Delta f/f$ (%)	*MAC*
B_1	1.211	1.191	−1.65	0.997	1.211	1.201	−0.83	0.982
B_2	1.289	1.260	−2.25	0.986	1.270	1.260	−0.79	0.955
T_1	3.564	–	–	–	3.525	3.467	−1.65	0.942
B_3	3.984	3.877	−2.69	0.870	3.984	3.906	−1.96	0.936
B_4	4.141	4.059	−1.98	0.804	4.141	4.063	−1.88	0.982

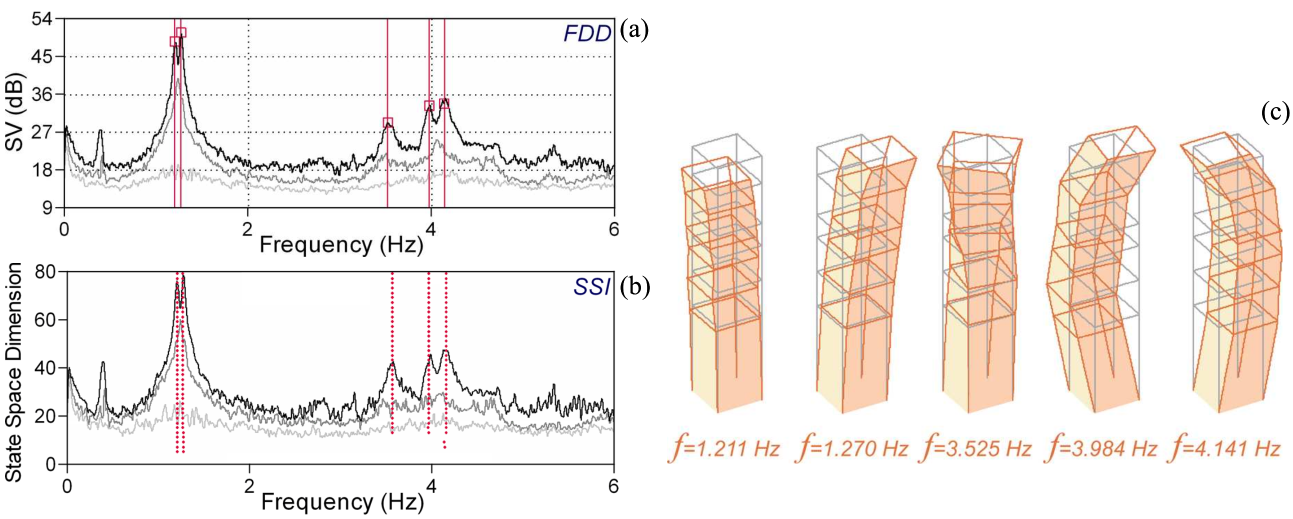


Figure 3: June 2008, data series AV1: (a) Singular Values of the spectral matrix and selected modes (FDD technique); (b) Stabilization diagram (SSI technique); (c) Vibration modes (FDD)

F.E. Modelling and Model Tuning

The experimental investigation was preceded by the development of a 3D finite element model (Fig. 4), based on the available geometric survey. The F.E. program SAP2000 was used to create the numerical model. The tower was modelled by using 8-node brick elements. A relatively large number of finite elements have been used in the model, so that a regular distribution of masses could be obtained and all the openings in the load-bearing walls could be reasonably represented. The model consists of 3481 solid elements with 17196 active degrees of freedom.

Since the geometry of the tower was accurately surveyed, the main uncertainties are related to the boundary conditions and the characteristics of the material. In order to reduce the number of uncertainties in the model calibration, the following assumptions were introduced: (a) the weight per unit volume of the masonry was assumed as 17.0 kN/m^3; (b) the Poisson's ratio of the masonry was held constant and equal to 0.15; (c) since the soil-structure interaction is hardly involved at the low level of ambient vibrations that existed during the tests, the tower footing was considered as fixed.

Subsequently. a sensivity analysis was carried out in order to explore the influence of the average elastic characteristics of stone masonry and of the connection (Fig. 1) between the tower and the neighbouring building on the available dynamic characteristics (natural frequencies and mode shapes).

Once the sensitivity analysis provided a confirmation on the correct choice of the updating parameters, the FE model was refined in 3 steps of systematic manual tuning, varying within each step the assumption of the material from isotropic (FEM1) to orthotropic (FEM2), and considering the contribution of the adjacent buildings by a spring series (FEM3). In the last step (FEM4), the uncertain structural parameters were identified in order to enhance the correlation between experimental and numerical modal behaviour using the technique described in (Douglas and Reid 1982).

In the manual tuning, the correlation between the dynamic characteristics of the FE models and the experimental results was evaluated via the maximum absolute frequency discrepancy $D_{F,max}$:

$$D_{F,\max} = \max(D_{F,i}) \tag{1}$$

$$D_{F,i} = 100\left|\frac{f_{FEM,i} - f_{FDD,i}}{f_{FDD,i}}\right| \tag{2}$$

and the average frequency discrepancy J (Table 2):

$$J = \frac{1}{M}\sum_{i=1}^{M} D_{F,i} \tag{3}$$

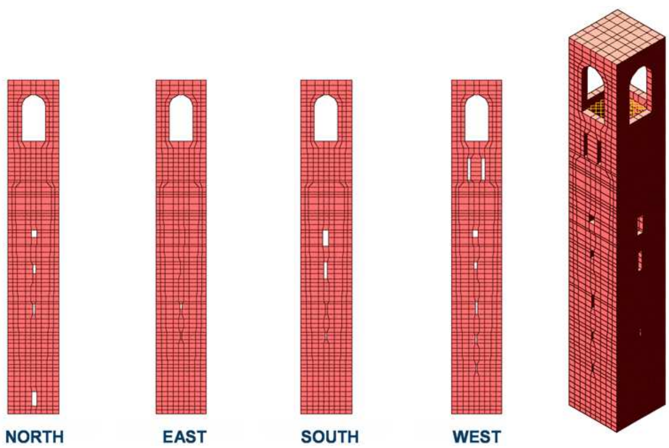


Figure 4: Finite Element model of the tower

FEM1 A preliminary dynamic analysis was performed to check the similarity between experimental and theoretical modal parameters. In this analysis, the Young's modulus of stone masonry was assumed equal to 3.00 GPa. It is worth noting that the assumed value was suggested either by engineering judgement or by the results of sonic tests while exhaustive tests to evaluate the mechanical characteristics of the stone masonry are not yet available.

The comparison between theoretical and experimental modal parameters shows highly imperfect correlation, since the average frequency discrepancy ranges up to about 21.63% with the maximum value of 57.99% (Table 2). Beyond that, FEM1 does not accurately represent the structural behaviour of the tower since:

a. the model is much stiffer than the tower (with all the natural frequencies of FEM1 model significantly exceeding the experimental ones);

b. the torsion mode T_1 of the model does not follow the experimental sequence, where the torsion mode is placed between two couples of bending modes; on the contrary, the torsion mode follows two couples of bending modes in FEM1 model;

c. also the mode shapes of bending modes exhibit major differences with the experimental results. Specifically, FEM1 bending modes involve motion along the main N-S and E-W directions while the identified tower modes involve bending along the diagonals (Fig. 3(c)).

FEM2 The poor quality of correlation clearly indicates that the assumptions on the isotropic behaviour of stone masonry and on the connection with the neighbouring building need to be revised. Hence, an orthotropic elastic behaviour (FEM2) was assumed for the stone masonry; the average characteristics of the material were E = 3.00 GPa, G_{13} = G_{23} = 0.33 GPa. The introduction of orthotropic elasticity dramatically improved the correlation with the experimental results (Table 2). In particular:

a. the stiffness of the model significantly decreased, so that the average and maximum frequency discrepancies are reduced from 21.63% to 8.52% and from 57.99% to 17.11%, respectively;

b. the torsion mode T_1 of the FEM2 model correctly follows the experimental sequence.

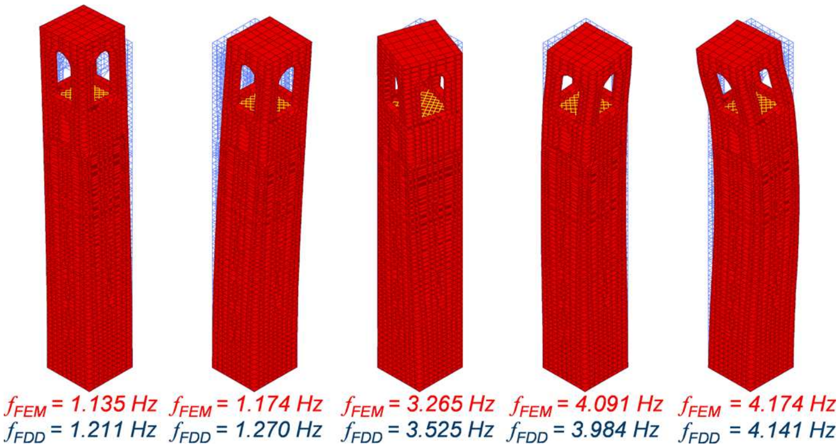
On the other hand, the flexural modes of FEM2 continue to exhibit bending along the main N-S and E-W directions, differently from the observed modes (involving bending along the diagonals).

FEM3 A third model FEM3 was subsequently developed, by accounting for the connection between the tower and the church through a series of linear (nodal) springs of constant k. After some manual tuning, the stiffness of springs $k = 4\times10^4$ kN/m was assumed. As it had to be expected from previous investigations (Gentile and Saisi 2007), now the bending modes are fully consistent with the experimental results; in addition, further reduction of frequency discrepancies was attained (Table 2), with J, the average frequency discrepancy less than 5.91%. Of course, the value of $D_{F,max}$ = 11.29% (obtained for mode T_1) remains quite high and the differences are surely to be related to the simplified distribution of the model elastic properties, which were held constant for the whole structure. However, the correlation between theoretical and experimental behaviour is satisfactory and provides a robust verification of the FEM3 model main assumptions, being a one-to-one correspondence between the mode shapes.

FEM4 The uncertain structural parameters (E, G_{13}=G_{23}, k) were finally estimated by minimizing the difference between theoretical and experimental natural frequencies through the procedure proposed by Douglas and Reid (1982). The optimizing iterative procedure identified more accurately the uncertain values concerning the material properties and the spring stiffness used in the updated model, as follows: E=2.64 GPa, G_{13}=G_{23}=0.37 GPa, k=3.33×10^4 kN/m. Once the optimal estimate of the parameters of FEM4 model was performed, a complete correlation analysis between the theoretical and experimental modal parameters was carried out (Fig. 5). The results shows a very good agreement of the model with the experimental results, being $D_{F,max}$ equal to 7.56% and particularly low for the higher bending modes, in spite of the simplified distribution of the elastic properties. Hence, the model seems suitable for a successive investigation, including further non-destructive tests on the materials and the application of more refined system identification techniques.

Table 2: Correlation between the experimental results and the computed frequencies

	B_1	B_2	T_1	B_3	B_4	J	$D_{f,MAX}$
FDD	1.211	1.270	3.525	3.984	4.141	–	–
FEM1	1.220	1.246	5.569	4.982	5.072	21.63	57.99
FEM2	1.123	1.141	2.922	3.876	3.919	8.52	17.11
FEM3	1.183	1.141	3.127	4.126	4.233	5.91	11.26
FEM4	1.135	1.174	3.265	4.091	4.174	4.94	7.56



$$D_{F,i} = 100 \cdot \left| \frac{f_{FEM,i} - f_{FDD,i}}{f_{FDD,i}} \right|$$

$$D_{F,\max} = \max(D_{F,i}) = 7.56$$

$$J = \frac{1}{M} \sum_{i=1}^{M} D_{F,i} = 4.94$$

Figure 5: FEM4: vibration modes and comparison with the experimental results

Conclusions

The paper focuses on the procedure developed for the calibration of the structural FE model of a bell tower by using a systematic model tuning. The tuning is based on the availability of the dynamic characteristics of the investigated structure (in terms of natural frequencies and mode shapes) and of the accurate survey of the geometry and of the crack pattern.

The research demonstrates the importance of the dynamic testing in the model tuning of historic structures. It is further noticed that the results obtained in the case study seem to indicate that the orthotropic elasticity assumption is more suitable to modelling stone masonry structures.

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