COMPDYN 2013
4th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, N.D.Lagaros, V. Plevris (eds.)
Kos Island, Greece, 12–14 June 2013

POST-EARTHQUAKE ASSESSMENT OF A MASONRY TOWER BY ON-SITE INSPECTION AND OPERATIONAL MODAL TESTING

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Keywords: Earthquake, Masonry Towers, On-site Survey, Operational Modal Testing

Abstract. *The paper describes the methodology applied to assess the state of preservation of the tallest historic tower in Mantua, the "Gabbia" tower, after the earthquakes of May 2012. An extensive investigation programme – including visual inspection, sonic and flat jack tests, operational modal testing and structural modelling – has been planned to support the future preservation actions of the tower. The paper focuses especially on the information collected during on-site survey and dynamic tests and describes how these results can be employed and cross-correlated to assess the structural condition and seismic vulnerability of the tower.*

1 INTRODUCTION

On May 29th, 2012 a strong earthquake occurred in Emilia-Romagna region but it was significantly felt also in Lombardia region and damages were reported on several historic buildings placed in the town of Mantua, where Politecnico di Milano has a large campus. Consequently, the VIBLAB (Laboratory of Vibration and Dynamic Monitoring of Structures) of Politecnico di Milano was committed to assess the structural condition and to evaluate the seismic performances of the taller historic tower of the city, the *Gabbia* tower [1]. The tower (Fig. 1), about 54.0 m high and surrounded by an important historic building, is a symbol of the cultural heritage in Mantua so that the fall of small masonry pieces from its upper part, reported during the earthquake, provided strong motivations for deeply investigating the seismic vulnerability of the building.

The multi-disciplinary approach planned to assess the structural safety and the seismic vulnerability of the *Gabbia* tower involves both experimental and analytical analysis, including several tasks [2-4]: (a) historic and documentary research; (b) geometric survey and field survey of the crack pattern; (c) non-destructive and slightly destructive tests of materials on site (i.e. sonic pulse velocity tests and flat-jack tests); (d) dynamic tests in operational conditions; (e) F.E. modelling and vibration-based validation of the model; (f) use of the validated model to asses the structural safety and predict the seismic performance, according to the provisions of the current Italian guidelines for the seismic risk mitigation of cultural heritage [5].

After a brief description of the *Gabbia* tower, the paper summarizes the information and the results provided by the execution of visual inspection and dynamic tests, performed between July 30th and August 3rd, 2012 with the support of a mobile platform (Fig. 1).

Figure 1: Views of the *Gabbia* tower in Mantua, Italy.

2 DESCRIPTION OF THE TOWER AND HISTORIC BACKGROUND

The *Gabbia* tower, about 54.0 m high, is the tallest tower in Mantua, overlooking the historic centre listed within the UNESCO Heritage (Figs. 1-3). The Tower, dating back to XIII century [1], is built in solid masonry bricks. An important palace surrounds the tower (Figs. 1-3); even if the building dates back nearly the same age of the tower, the load bearing walls of the palace are not effectively linked but just drawn.

The tower has an almost squared plan and the load bearing walls are about 2.4 m thick until the upper levels (Fig. 3). Precious frescoes, dating back to XIV and XVI centuries, decorate the tower's fronts embedded in the palace, whereas in 1811 the interior walls were painted with refined decorations [1]. The top part of the building has a two level lodge, which hosted in XIX century the observation and telegraph post reachable by a wooden staircase, no more practicable since several years.

In the XVI century, the *Gabbia* tower was used as open-air jail, hosting a hanged dock on the S-W front (Figs. 1 and 3). Few information is available on the building evolution but the observation of the masonry texture reveals passing-through discontinuities at the top. Traces of past structures are visible on all fronts (Fig. 4) and the presence of merlon-shaped discontinuities (Figs. 4 and 5) suggests modifications and further adding at the top of the tower. Moreover, at about 8.0 m from the top, a clear change of the brick surface workmanship (the bricks of the lower part are superficially scratched) could reveal a first addition (Fig. 6); in the same region concentrated changes of the masonry texture reveal local repair.

The inner access to the tower was re-established recently (October 2012) through provisional scaffoldings; it should be noticed that the entrance is at 17.7 m from the ground level (Fig. 3) and the access to the lower portion and the base of the building is not possible.

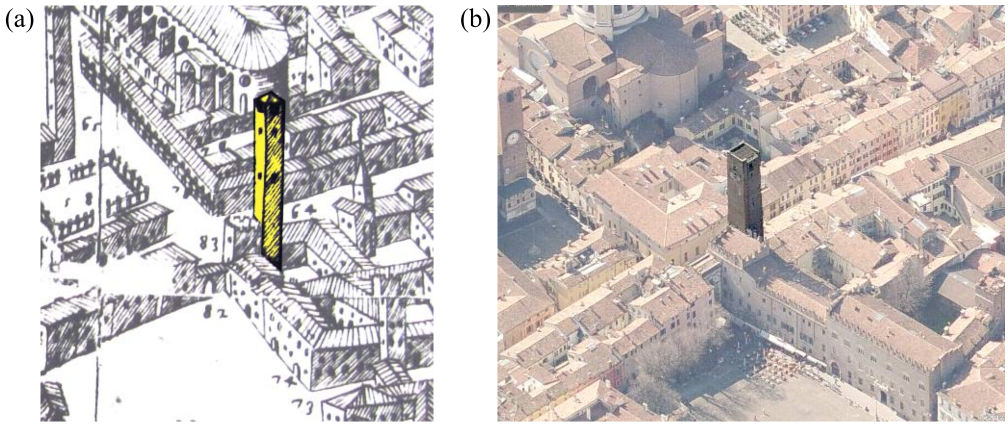


Figure 2: The *Gabbia* tower and the surroundings: (a) view of the XVII century [6]; (b) recent view.

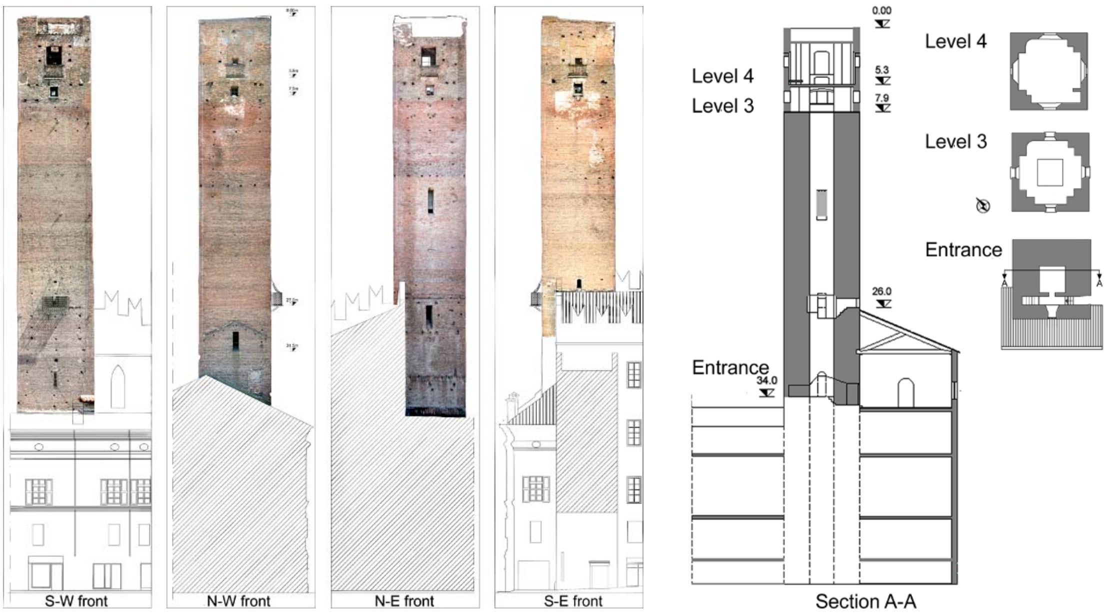


Figure 3: Fronts and section of the tower.

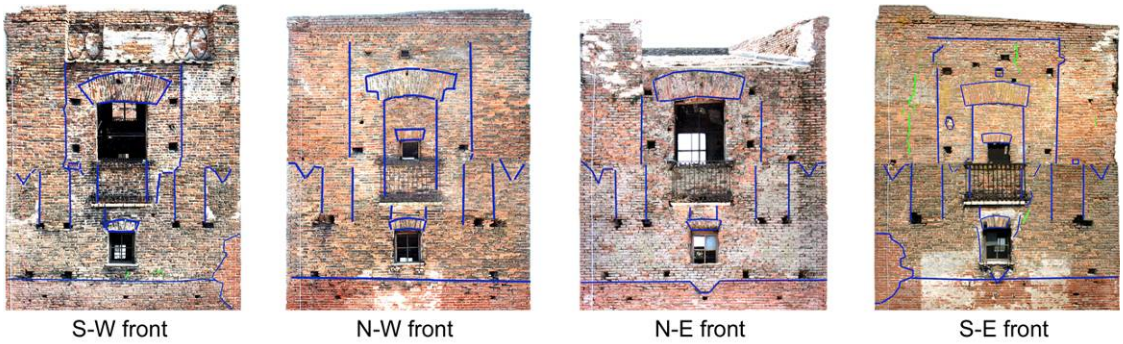


Figure 4: Map of the discontinuities on top of the tower.

Figure 5: Probable merlons embedded in the masonry texture.

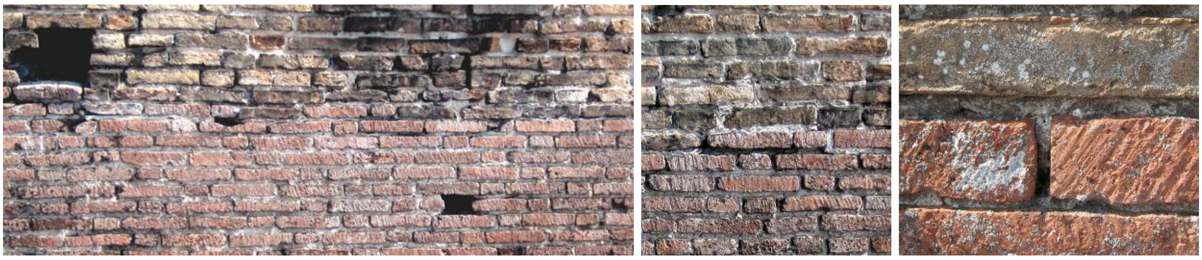


Figure 6: Change of the surface workmanship at about 8.0 m from the top (the bricks of the lower portion are superficially scratched).

3 VISUAL INSPECTION AND ON SITE TESTS

As previously pointed out, an accurate on-site survey of all fronts of the tower was firstly performed using a mobile platform (Fig. 1). This preliminary survey is aimed at providing details on the geometry of the structure and identifying critical areas and irregularities, where more refined inspections are needed. In the meantime, the historical evolution of the structure should be known to explain the signs of damage detected on the building.

Excepting the upper part of the tower, visual inspection did not reveal evident structural damage but only superficial decay of the materials (mainly of the mortars, due to the natural ageing and the lack of maintenance). In the lower part of the tower, the corners exhibit rare thin cracks and the masonry section appears of solid bricks and lime mortars. Subsequent pulse sonic tests and double flat jacks confirm the soundness and the compactness of the masonry of such building portion. Results from pulse sonic tests suggest solid brick section, with high velocity values, ranging between 1100 m/s to 1600 m/s. Double flat jack test carried out on the N-E side at about 32.8 m from the ground level (Fig. 7) revealed that the Young's modulus is larger than 3.00 GPa. Similar information result from the laboratory tests on sampled bricks and mortars.

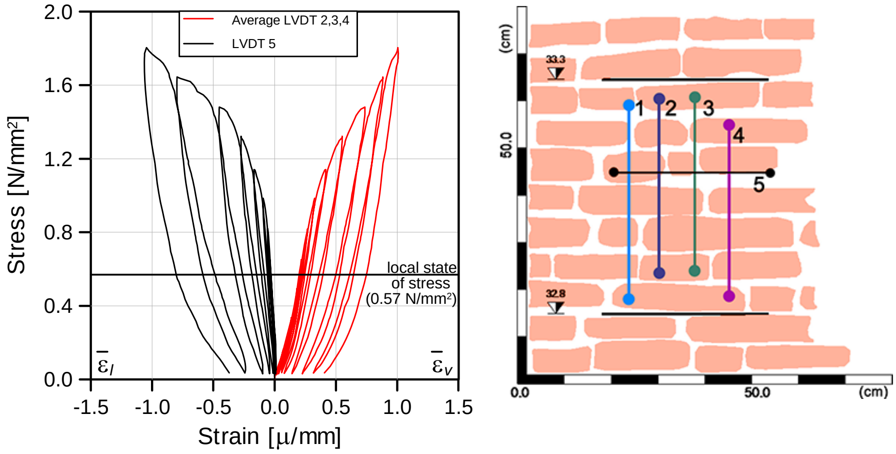


Figure 7: Results of the double flat jack test and superficial texture of the investigated area.

Figure 8: Typical discontinuities on top of the tower and lack of horizontality of the masonry courses.

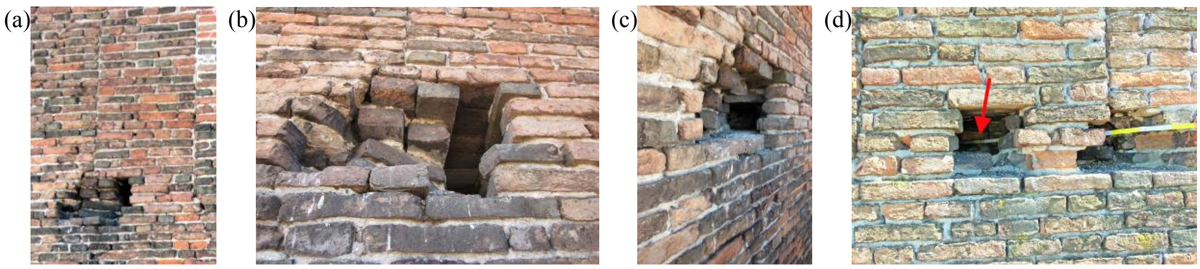


Figure 9: Details of the infillings between the supposed merlons on the N-W front: (a), (b), (c) left side infilling; (d) right side infilling.

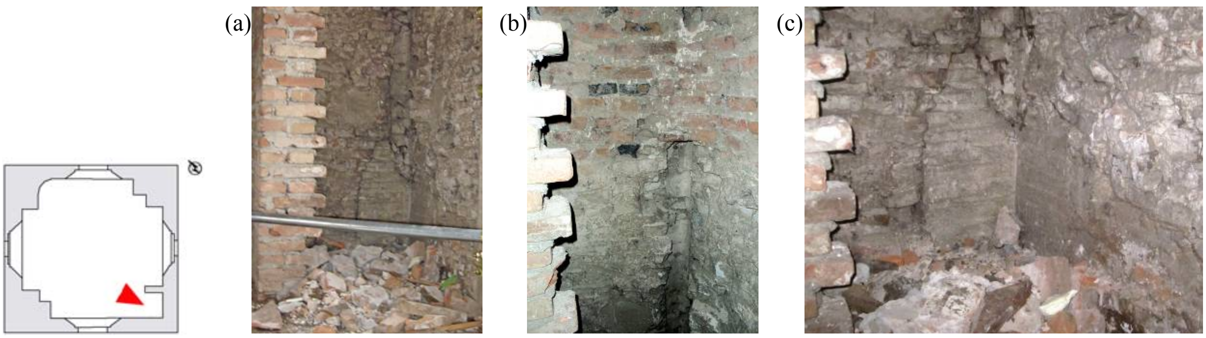


Figure 10: Details of the South corner at level 4: (a) the corner is partially dismantled, (b) with an embedded pipe at the edge and (c) a probable merlon.

On the contrary, the top of the tower (i.e. the upper portion about 8.0 m high, Fig. 3) shows damage related to the detachment of the several construction phases, worsened by the natural decay (Fig. 4). In fact, settlement of the interventions and of the opening infillings coupled with the highly disordered masonry and mortar erosion causes lack of horizontality of the joints of the crowing and the development of cracks as well (Fig. 8).

Critical areas are the infillings between merlons (Figs. 4 and 9), supported only by few courses of thin masonry due to the unusual layout of the scaffolding holes. The extension of the scaffolding holes beside the base of the infillings weakens the local and overall stability (Fig. 9) so that the prevention of local instability of such masonry portions should be one of the intervention priority on the tower.

It is important to remark the change of the masonry sections and of the plan layout in the upper part (Levels 3 and 4 in Fig. 3), where the nearly squared plan turns into corner masonry piers and un-toothed infillings. The decrease of resisting section is especially significant for the piers on North and South corners at level 4. As shown in Fig. 10, the corner pier at South is partially dismantled, showing an embedded pipe at the edge and the merlon shape. The lack of any mortar encrustation in the merlon surface suggests a weak connection in the other piers at the same level, as well.

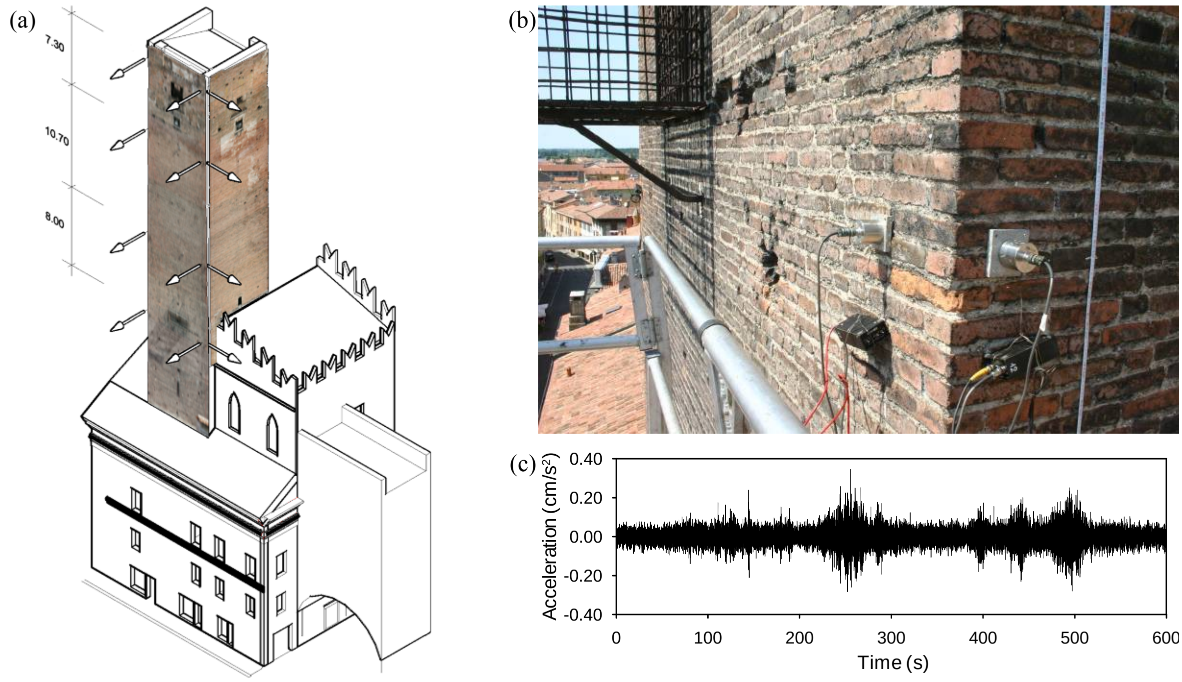


Figure 11: (a) Sensor layout adopted in ambient vibration testing of the *Gabbia* tower (dimensions in m); (b) typical mounting of accelerometers; (c) sample of acceleration recorded at the top of the tower

4 AMBIENT VIBRATION TESTS AND MODAL IDENTIFICATION

4.1 Testing procedures and modal identification

Ambient vibration tests (AVTs) were conducted between July 31st and August 2nd 2012 using a 24-channel data acquisition system (24-bit resolution, 102 dB dynamic range and anti-aliasing filters) and piezoelectric accelerometers (WR model 731A, 10 V/g sensitivity and ±0.50 g peak). A short cable (1 m) connected each sensor to a power unit/amplifier (WR model P31), providing the constant current needed to power the accelerometer's internal amplifier, signal amplification and selective filtering.

The response of the tower was measured in 12 selected points, belonging to 4 different cross-sections along the height of the building, according to the sensor layout shown in Fig. 11a. Figure 11b shows two accelerometers mounted on the corner of the lower instrumented cross-section, corresponding to the level of the hanging dock on the S-W front (about 28.0 m from the top). It should be noticed that the positioning of the accelerometers at the upper levels was aimed at checking if the change of masonry texture detected in visual inspection (and the thickness decrease of load bearing walls, Fig. 3) affects the dynamic characteristics of the tower. Hence, the upper instrumented sections (Fig. 11a) were at the crowning (about 2.0 m from the top) and just below the change of masonry texture (about 9.3 m from the top).

The excitation was only provided by wind and micro-tremors and acceleration data were acquired for 28 hours: from 16:00 to 23:00 of July 31st 2012 and between 9:00 of August 1st to 6:00 of August 2nd 2013. Figure 11c shows a sample of the acceleration recorded the upper instrumented level: it should be noticed that very low level of ambient excitation was present during the tests, with the maximum recorded acceleration being always lower than 0.4 cm/s^2.

The modal identification was performed using time windows of 3600 s. The sampling frequency was 200 Hz, which is much higher than that required for the investigated structure, as the significant frequency content of signals is below 12 Hz. Hence, low pass filtering and decimation were applied to the data before the use of the identification tools, reducing the sampling frequency from 200 Hz to 40 Hz; after decimation, the number of samples in each 1-hour record was of 144000, with a sampling interval of 0.025 s.

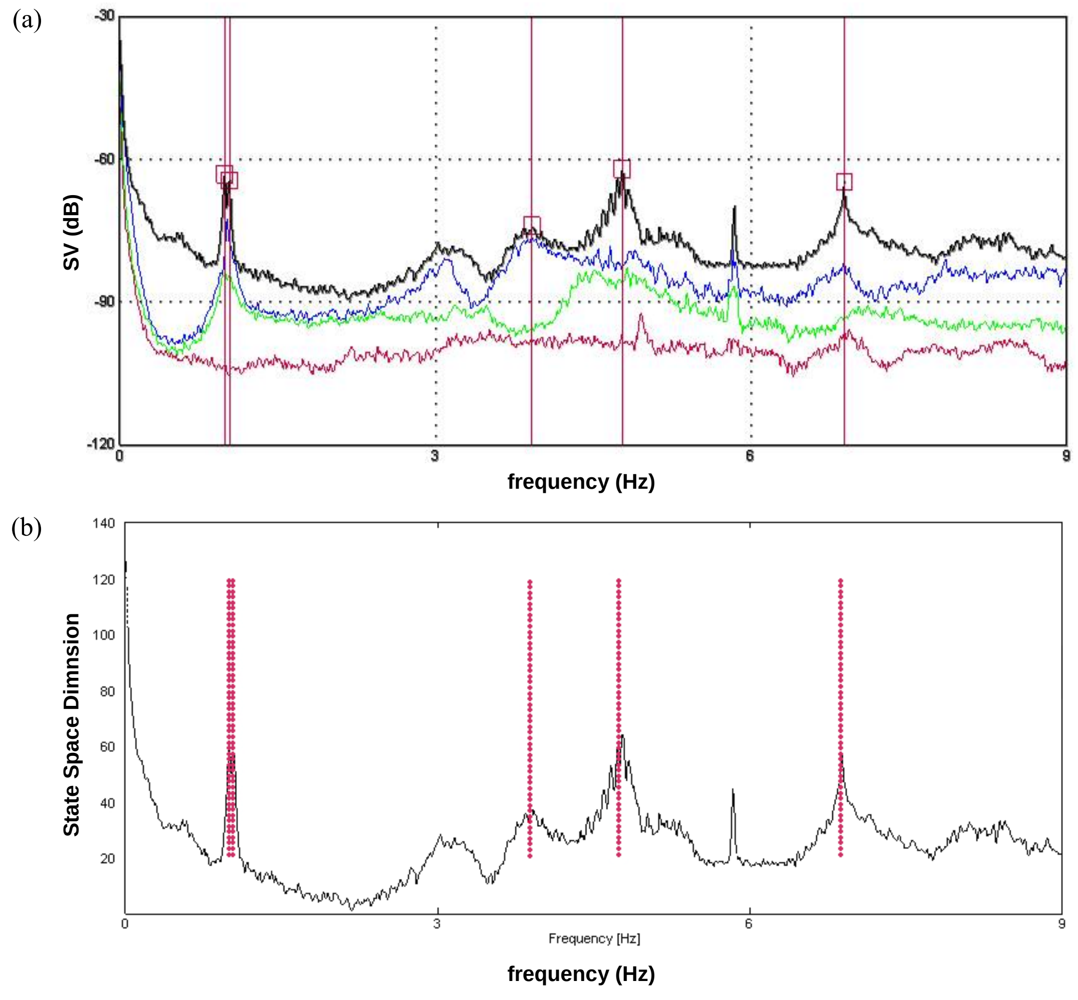
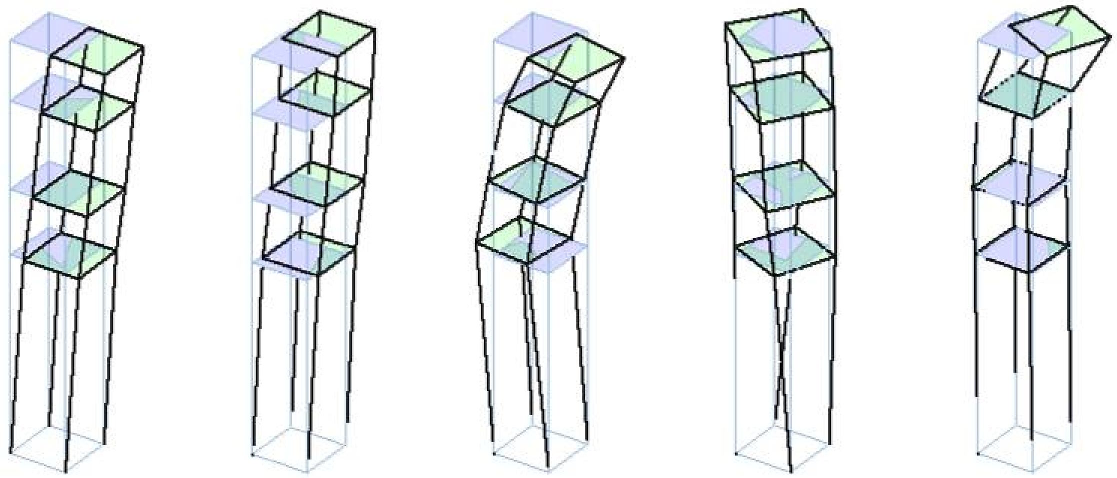


Figure 12: Dataset recorded on July 31st 2013, 16:00-17:00: (a) Singular value (SV) lines and identification of natural frequencies (FDD); (b) stabilization diagram (SSI).

The extraction of modal parameters from ambient vibration data was carried out by using the Frequency Domain Decomposition (FDD) technique [7] in the frequency domain and the data driven Stochastic Subspace Identification (SSI) method [8-9], in the time domain; these techniques are available in the commercial software ARTeMIS [10]. More specifically, the FDD was mainly applied on site in order to quickly estimate the dynamic characteristics of the structure, whereas back in the office those estimates were refined using the SSI.

During the dynamic tests, a second acquisition system was used to measure the temperature in three different points of the tower: on the S-W front both indoor and outdoor temperature were measured, whereas only the outdoor temperature was measured on the S-E front. It is worth mentioning that the changes of outdoor temperature were very significant and ranged between 25°C and 55°C, whereas slight variations were measured by the indoor sensor (29°C-30°C) due to the high thermal inertia of the load bearing walls.



B1) f_{SSI} = 0.993 Hz B2) f_{SSI} = 1.034 Hz B3) f_{SSI} = 3.916 Hz T1) f_{SSI} = 4.770 Hz L1) f_{SSI} = 6.873 Hz

Figure 13: Vibration modes generally identified from ambient vibration measurements (SSI, July 31st 2013, 16:00-17:00).

4.2 Dynamic characteristics of the Tower

Notwithstanding the very low level of ambient excitation (Fig. 11c) that existed during the tests, the application of both FDD and SSI techniques to all collected data sets generally allowed to identify 5 vibration modes in the frequency range of 0-7 Hz.

Typical results in terms of natural frequencies and mode shapes are shown in Figs. 12 and 13, respectively. The plots in Figs. 12 and 13 refer to the acceleration data recorded in the time window 16:00-17:00 of July 31st 2013. The first Singular Values (SV) of the spectral matrix resulting from the application of the FDD technique is shown in Fig. 11a, whereas Fig. 11b shows the stabilization diagram obtained by using the SSI method; the corresponding mode shapes, identified via SSI, are shown in Fig. 13.

The inspection of Figs. 12 and 13 allows the following comments on the dynamic characteristics of the *Gabbia* tower:

(a) two closely spaced modes were identified around 1.0 Hz. These modes are dominant bending (B) and involve flexure in the two main planes of the tower, respectively: the mode B1 (Fig. 13B1) is dominant bending in the N-E/S-W plane whereas the modal deflections of B2 (Fig. 13B2) belong to the orthogonal N-W/S-E plane;
(b) the third mode B3 (Fig. 13B3) involves bending in the N-E/S-W plane, with slight (but not negligible) components in the orthogonal plane;
(c) just one torsion mode (T) was identified (Fig. 13T1) and the corresponding natural frequency was 4.77 Hz (in the examined time window);
(d) the last identified mode is local (L) and only involves deflections of the upper portion of

the tower (Fig. 13L1). The mode shapes seems dominant bending, with significant components along the two main planes of the tower. The presence (and generalized detection) of a local vibration mode provides further evidence of the structural effect of the change in the masonry quality and morphology (including un-toothed opening infillings and discontinuities) observed on top of the tower during the preliminary visual inspection. On the other hand, both visual inspection and OMA confirm the concerns about the seismic vulnerability of the buildings and explains the fall of small masonry pieces from the upper part of the tower, reported during the earthquake of May 29th 2013.

4.3 Frequency variation and correlation with outdoor temperature

Statistics of the natural frequencies that were identified between 31/07/2012 and 02/08/2012 are summarized in Table 1 through the mean value, the standard deviation, the extreme values and the coefficient of variation of each modal frequency. It should be noticed that the natural frequencies of all modes exhibit slight but clear variation, with the standard deviation ranging between 0.011 Hz (mode B2) and 0.037 Hz (mode L1).

Mode	f_{ave} (Hz)	σ_f (Hz)	f_{min} (Hz)	f_{max} (Hz)	CV (%)
1 (B1)	0.981	0.018	0.957	1.014	1.826
2 (B2)	1.026	0.011	1.006	1.052	1.093
3 (B3)	3.891	0.025	3.857	3.936	0.654
4 (T1)	4.763	0.022	4.714	4.802	0.462
5 (L1)	6.925	0.037	6.849	6.987	0.528

B = bending mode; T = torsion mode; L = local mode

Table 1 Statistics of the natural frequencies identified during 28 hours of dynamic testing

(a)

(b)

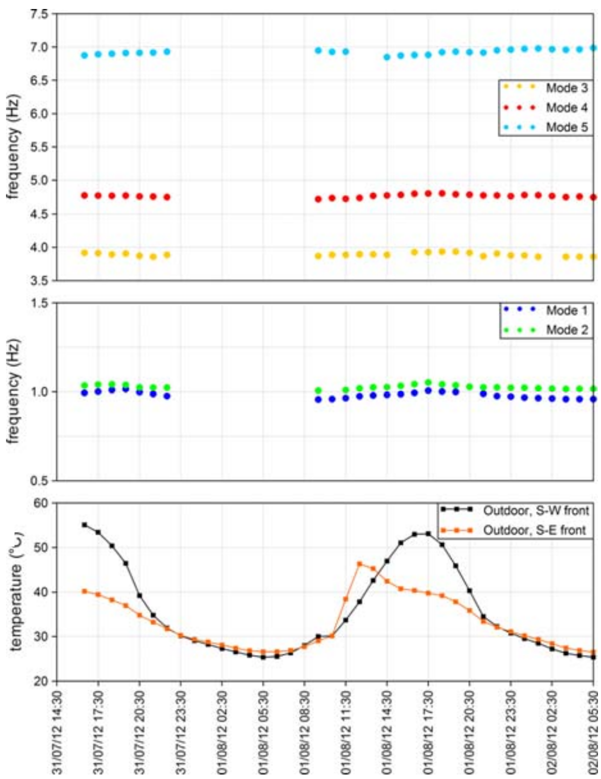


Figure 14: Variation in time of: (a) the natural frequency of identified vibration modes (SSI); (b) the temperature measured outdoor on the fronts S-W and S-E.

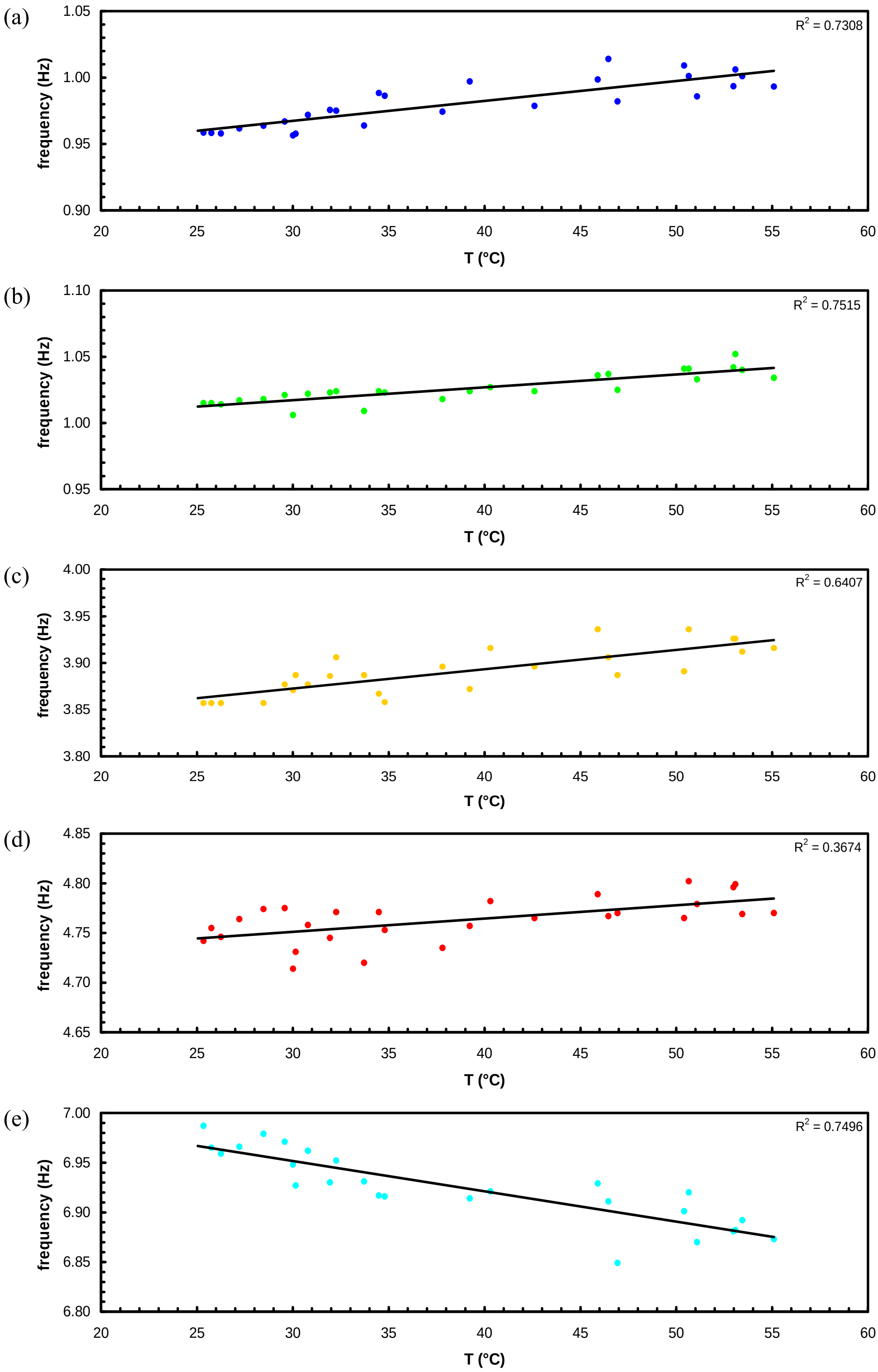


Figure 15: Frequency (SSI) variation versus measured outdoor temperature (S-W front): (a) mode B1; (b) mode B2; (c) mode B3; (d) mode T1; (e) mode L1.

Due to the very low amplitude of ambient excitation that existed during the 28 hours of acquisition, the variation of natural frequencies is conceivably related to the environmental (i.e. temperature) effects. In order to investigate the possible relationships between natural frequencies and temperature, Fig. 14 shows the evolution of natural frequencies, identified via

SSI, and the hourly averaged temperature, measured outdoor on the fronts S-W and S-E. The inspection of Fig. 14 suggests that:

1. the natural frequencies of the lower modes (global modes B1-B3 and T1) seem to increase with increased temperature. This behaviour, observed also in past experiences [4], [11] on masonry structures, can be explained through the closure of superficial cracks, masonry discontinuities or mortar gaps induced by the thermal expansion of materials. Hence, the temporary "compacting" of the materials induces a temporary increase of stiffness and modal frequencies, as well;
2. for the higher mode identified (that is a local mode of vibration), the variation in time of the natural frequency seems quite different.
3. the oscillation in time of natural frequencies of lower modes seems almost perfectly in-phase with the outdoor temperature on the S-W front.

In order to better explore the frequency-temperature correlation, the SSI estimates of all modal frequencies are plotted versus the outdoor S-W temperature in Figs. 15a-e. Each graph also shows the regression line and the coefficient of determination R^2.

R^2 is a dimensionless parameter and measures the variation percentage of Y (i.e. the k-th modal frequency) caused by the X variation (i.e. the temperature). The coefficient of determination ranges from 0 to 1; a value of 1 implies that the statistical model fits perfectly the experimental data whereas R^2 is equal to 0 in absence of correlation between the model and the experimental data.

Figures 15a-c confirm that the natural frequencies of bending modes B1-B3 increase as temperature increases and exhibit an almost linear correlation with the temperature, with R^2 ranging between 0.64 (mode B3, Fig. 15c) and 0.75 (mode B2, Fig. 15b).

The frequency-temperature correlation seems less strong for the torsion mode T1 (Fig. 15d) since R^2 is quite low (0.37) in comparison with the values of the other modes.

Nevertheless, it can be observed that the natural frequencies of all global modes clearly tend to increase with increased temperature.

Unlike the global modes, the natural frequency of the local mode L1 (Figure 15e) decreases as the temperature increases and the coefficient of determination assumes a high value ($R^2 \cong 0.75$). This result suggests that the thermal expansion of materials in a very inhomogeneous area of the structure causes a general worsening of the connection between the masonry portions; hence, further evidence seems to be provided, again in agreement with the main observation of the visual inspection, of the poor state of preservation of the upper part of the tower.

5 CONCLUSIONS

The paper demonstrates the importance of a multi-disciplinary approach in the assessment of historic buildings and especially in prompt post-earthquake investigation. The available information from visual inspection and on-site survey, suggesting the possible building evolution and indicating the masonry changes, provides a preliminary knowledge of utmost importance for planning the sensor layout in dynamic tests as well as for the execution of further tests.

Visual inspection of all main bearing walls clearly indicated that the upper part of the *Gabbia* tower is characterized by the presence of several discontinuities due to the historic evolution of the building, local lack of connection and extensive masonry decay.

The poor state of preservation of the same region was confirmed by the observed dynamic characteristics. In particular, the higher mode identified (having average frequency of 6.93 Hz) turned out to be local and involves only the top portion of the tower; the corresponding mode shapes is dominant bending, with significant components along the two main planes of the tower. It is further noticed that the local mode was clearly identified by applying different output-only techniques to the ambient response data collected for more than 24 hours on the historic structure.

The presence of a such local mode highlights the relevant structural effects of the change in the masonry quality and morphology observed on top of the tower in the preliminary visual inspection. Furthermore, the natural frequency of the local mode clearly decreases as temperature increases, suggesting that the thermal expansion of materials in a very inhomogeneous area of the structure, causes a general decrease of the connection between the masonry portions.

ACKNOWLEDGEMENTS

The investigation was supported by the Mantua Municipality. M. Antico, M. Cucchi (VIBLAB, Laboratory of Vibrations and Dynamic Monitoring of Structures, Politecnico di Milano) and Eng.. Marco Guidobaldi are gratefully acknowledged for the assistance during the field tests.

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