Post-earthquake diagnostic investigation of a historic masonry tower

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1. Introduction and research aims

The international debate on Cultural Heritage preservation generally agrees on the major role played by effective diagnostic survey in the structural assessment of historic buildings [1]. The diagnostic phase (i.e. the evaluation of the current health condition) – performed by collecting information on the characteristics of the building, the properties of materials, the historic transforma-tion of the structures and existing damage – provides a sound basis for any further evaluation of the safety level [2] as well as for the definition of appropriate intervention measures.

The collected information should include accurate investigation of the actual geometry, survey of the crack pattern and local visual inspections: these tasks, joined to historic research, provide a first interpretation of the structural layout and reveal the presence of masonry discontinuities (generally associated to transformations of the building and/or repair), possible vulnerabilities and age-ing issues. Visual inspections also suggest the positions that are more suitable for non-destructive (ND) or minor destructive (MD) tests and material sampling, necessary to evaluate the character-istics of the masonry and to explore local defects. Of course, the extension and deepening of the diagnostic phase is closely related to the observed local and global conditions of the building.

In the process of Architectural Heritage preservation, a still open issue is the linking between the information locally collected and the overall structural behaviour, especially in complex structures evolved over time. Within this context, ambient vibration testing (AVT) and operational modal analysis (OMA, i.e. the identification of modal parameters from ambient vibration responses) seem the most effective tools to support the structural assessment since they are capable of collecting information on the global modal charac-teristics (i.e. natural frequencies, mode shapes and modal damping ratios) [3–13] and might identify the presence and the position of damage [6,14,15]. In addition, AVT is a fully ND test, especially suit-able to Cultural Heritage structures since the test is performed by just measuring the response to ambient excitation (micro-tremors, wind, etc.).

It should be noticed that AVT could be employed in prompt surveys (e.g. post-earthquake emergency) by using either traditional accelerometers mounted on outdoor walls or innovative non-contact sensors [16,17]. Furthermore, the results of AVT should represent the starting point of long-term dynamic monitoring [6,11,13] in order to perform vibration-based damage assessment or to evaluate the effects of repair interventions.

The paper presents the main results of a recent post-earthquake assessment of a masonry tower. The investigated tower (Fig. 1), about 54.0 m high and dating back to the XII century, is known as Gabbia Tower [18,19] and is the tallest tower in Mantua, Italy. The tower is a symbol of the Cultural Heritage in Mantua so that the fall of small masonry pieces from its upper part, reported during the

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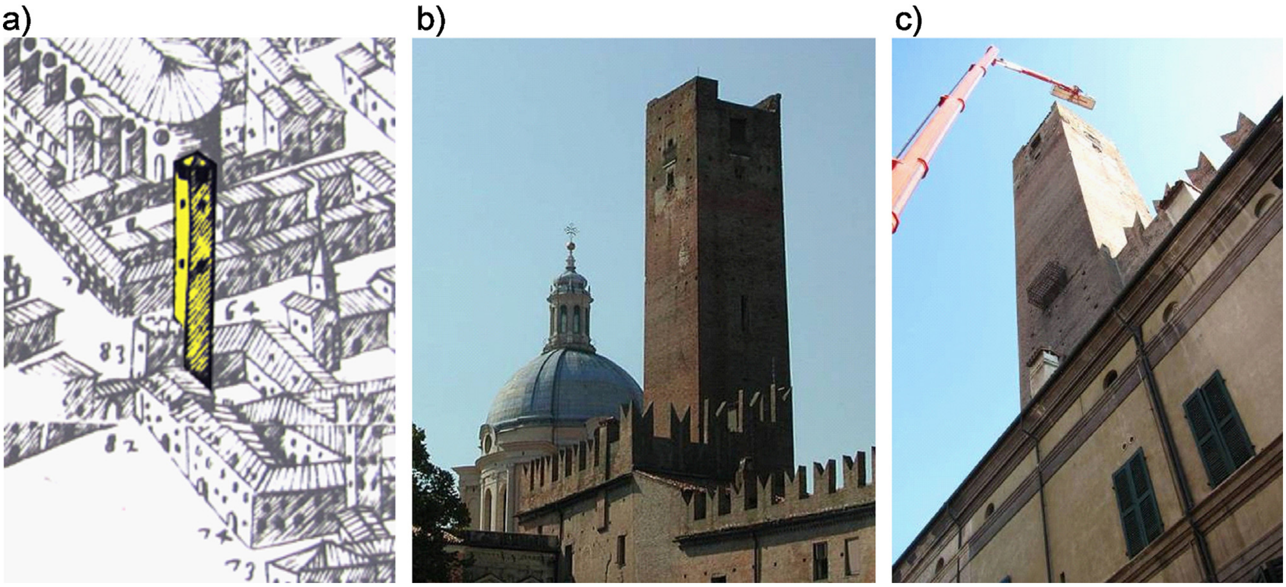

Fig. 1. The Gabbia Tower in Mantua, Italy. a: view of the XVII century [15]; b: recent view; c: view during the inspection (31/07/2012).

earthquake of 29/05/2012, provided strong motivations for deeply investigating the state of preservation and the seismic vulnerability of the building. Hence, an extensive research program was planned and performed to evaluate the structural condition of the tower.

After a brief description of the Gabbia Tower, the paper presents the results of the first part of the research, including: (a) historic and documentary research; (b) geometric survey and visual inspection of the bearing walls; (c) on-site survey of the crack pattern and structural discontinuities; (d) non-destructive and slightly destructive tests of materials on site; (e) ambient vibration tests.

The investigation is especially aimed at highlighting the key role of "global" testing methods, such as visual inspection and dynamic tests, in the diagnostic assessment of masonry towers. In the presented case study, visual inspection of all load-bearing walls clearly indicated that the upper part of the 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 and one local mode involving the upper part of the tower was clearly identified by applying different OMA techniques to the response data collected for more than 24 hours on the historic building.

2. Description of the tower and historic background

The Gabbia Tower [18], about 54.0 m high, is the tallest tower in Mantua, overlooking the historic centre listed within the UNESCO Heritage (Figs. 1 and 2). As it can be observed in Fig. 1, it is part of the complex architecture of an important palace, evolved since the XIII century around the tower. Precious frescoes, dating back to XIV and XVI centuries, decorate the tower's fronts embedded in the palace, whereas in 1811 the interior walls of the tower were painted with refined decorations [19].

The tower has been of private ownership for long time and only in the 1980s it passed to the Mantua Municipality.

The tower, built in solid brick masonry, has nearly square plan and the load-bearing walls are about 2.4 m thick until the upper levels (Fig. 2), where the corner masonry section decreases to about 0.7 m. The top part of the building has a two level lodge, which hosted the observation and telegraph post, used for military and communication purposes during the XIX century and at the beginning of the XX century. A wooden staircase reached the lodge but it was no more in function since the 1990s, due to the lack of maintenance. The inner access to the tower was re-established only recently (October 2012) through provisional scaffoldings in order to allow visual inspection and geometric survey of the inner load-bearing walls.

Few historic documents are available on the tower and its evolution [19]. Despite the foundation date is unknown, some recent research dates it back to the late XII century and assumes that the construction was probably concluded in 1227. The tower was part of the defensive system of the Bonacolsi family, which governed Mantua at the time. According to the past building tradition of defensive structures, the entrance is not at the ground level but at a higher position. At present, the entrance to the tower is at about 17.7 m (Fig. 2) and the access to the lower portion and to the base of the building is not possible. 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 2).

As previously pointed out, during the centuries a palace progressively evolved around the tower, complicating the geometry of the structure and the mutual links between the walls. In general, the load-bearing walls of the palace seem not effectively linked but just drawn to the tower's masonry walls, while the tower supports directly several floors and vaults.

No extensive information is available on the tower past interventions but the stratigraphic survey and the observation of the masonry texture reveals passing-through discontinuities in the upper part of the building, with those discontinuities being conceivably referable to the tower evolution phases. Traces of past structures are visible on all fronts (Fig. 3) and the presence of merlon-shaped discontinuities (Figs. 3 and 4) suggests modifications and successive adding in the upper part 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) suggests a first addition (Fig. 5); in the same region concentrated changes of the masonry texture reveal local repair.

A first hypothesis, based on the surface discontinuity survey (Fig. 3), could recognize the following construction phases (which are very difficult to date): (i) erection of the main building until the height of 46 m (probably concluded in 1227); (ii) subsequent addition until the merlon level; (iii) adding of 4 corner piers

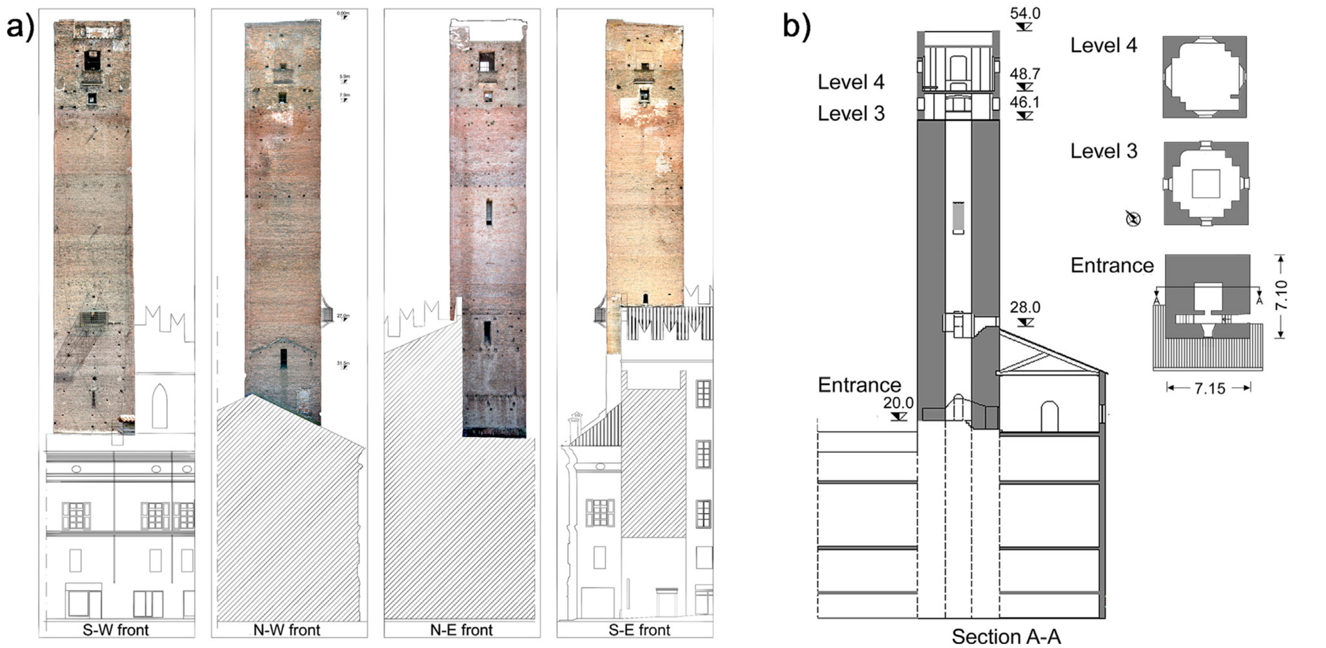

Fig. 2. Fronts and section of the tower (dimensions in m).

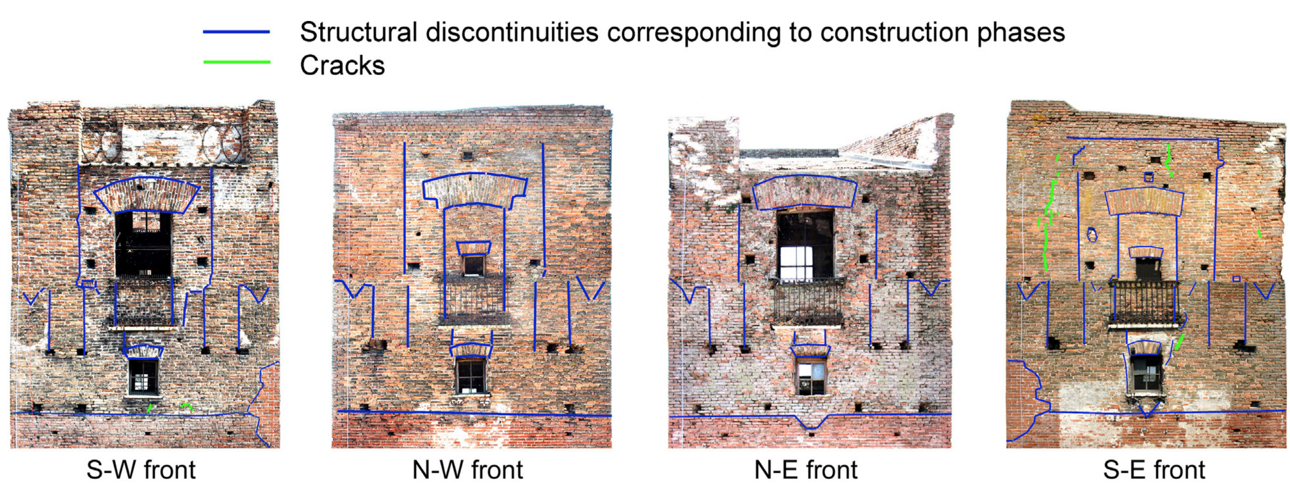

Fig. 3. Map of the discontinuities on top of the tower.

Fig. 4. Probable merlons embedded in the masonry texture.

Fig. 5. Change of the surface workmanship at about 8.0 m from the top.

supporting a four side roof; (iv–v) opening infilling and construction of windows, crowning and new roof; (vi) repair of the South corner.

3. Visual inspection and on-site tests

After the earthquakes on May 2012, accurate on-site survey of all outer fronts of the tower was firstly performed between 30/07/2012 and 02/08/2012, using a movable platform (Fig. 1c). As previously pointed out, survey of the inner bearing walls and local tests were performed lately (November 2012), once the inner access was re-established.

On-site survey was aimed at both providing details on the geom-etry of the structure and identifying critical areas, where more refined inspection is needed. At the same time, visual inspection and stratigraphic survey provides an important support to historical research by identifying undocumented interventions as well as the regions where masonry is homogeneous and/or characterized by discontinuities. This survey of masonry textures provides the evidence of local damages and is crucial especially in detecting local vulnerabilities and possible overturning mechanisms of unlinked masonry portions within a seismic assessment framework. Moreover, visual inspection triggers the subsequent investigations.

In present case, on-site survey highlighted two different structural conditions that were associated to:

- the main part of the building, until the height of about 46 m from the ground level;
- the upper part of the tower (Figs. 2 and 3).

Visual inspection of the outer bearing walls did not reveal evident structural damage in the lower part of the tower, where only superficial decay of the materials (i.e. mainly erosion of the mortar joints) was observed, due to the natural ageing and the lack of maintenance. Rare thin cracks were detected in the corners and the masonry section turned out to consist of solid bricks and lime mortars.

After the installation of a provisional metallic scaffolding inside the tower, pulse sonic tests and double flat-jack tests were performed and confirmed the compactness of the masonry in the lower portion of the tower. Results from pulse sonic tests provided the evidence of solid brick section, with sonic velocity values ranging between 1100 m/s and 1600 m/s. Double flat-jack tests were carried out: (a) on the outer S-W wall, at the ground level and (b) on the inner S-W wall (from scaffolding), at about 32.8 m from the ground level. The Young's modulus obtained from both tests turned out to be larger than 3.00 GPa. Similar information on the good quality of masonry materials results from the laboratory tests on sampled bricks and mortars.

On the contrary, the upper part of the tower (i.e. the upper region about 8.0 m high, Fig. 2) exhibited damage related to the abovemen-tioned detachment of several construction phases, worsened by the natural decay (Figs. 3 and 4). The poor state of preservation of the masonry is confirmed by the low average value of the sonic velocity (600 m/s) obtained from pulse sonic tests.

The infillings between merlons (Figs. 4 and 6) are conceivably the most critical areas because there is an almost complete lack of connection between the infillings and the neighbouring portions of the same wall; in addition, the unusual layout and the extension of the scaffolding holes (Figs. 4 and 6) at the base of the infillings often significantly weakens the local resistance, so that the preven-tion of local overturning mechanism should be one of the priority interventions.

It is worth underlining that the local structural issues in the top part of the building are worsened by the change in the masonry resisting section shown in Fig. 2: at the upper levels 3 and 4 (Fig. 2), the nearly square plan, about 2.4 m thick, turns into corner masonry piers and untoothed infillings. The section decrease is especially significant at level 4 for the piers on North and South corners. Furthermore, the pier on South corner is partially dismantled and contains a merlon shape as well as an embedded water pipe. In addition, the lack of any mortar encrustation in the merlon surface could suggest weak connection in all the piers at the same level.

It is marginally noticed that thermo-vision inspections were car-ried out on several walls of the tower portion embedded in the surrouning palace. The inspections were aimed at investigating the links between the walls and at confirming the presence of various infilled niches and openings, not represented in the available draw-ings of the building. In general, the lack of connection between the tower and the palace is clearly indicated by the presence of sharp cracks at the boundaries. Furthermore, a wide portion infilled by hollow bricks was found at the first floor (i.e. at about 5.0 m from the ground level): according to the flat owner statement, a room obtained in the masonry section was infilled when the tower's ownership was transferred to the Mantua Municipality.

4. Ambient vibration tests and modal identification

4.1. Testing procedures and modal identification

Taking profit of the availability of the movable platform (Fig. 1c) used during the inspection of outer walls, a first series of AVTs were conducted on the tower between 31/07/2012 and 02/08/2012.

Assuming that the building should have been excited by low-level ambient vibrations because the Mantua historic centre is closed to road traffic, the acceleration responses of the tower were measured using high sensitivity accelerometers (WR model 731A, 10 V/g sensitivity and ± 0.50 g peak). This accelerometer utilizes the piezoelectric effect of quartz to convert acceleration directly into a low impedance voltage signal. The frequency range for the accelerometer is 0.05 Hz to 500 Hz, which far exceeds the typical frequency range of interest for dynamic characterization

Fig. 6. Details of the infillings between the merlons on the N-W front.

of a masonry tower. 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. Two-conductor cables connected the amplifiers to a 24-channel data acquisition system (24-bit resolution, 102 dB dynamic range and anti-aliasing filters).

The response of the tower was simultaneously measured in 12 selected points, belonging to 4 different cross-sections along the height of the building, according to the single set-up shown in Fig. 7. 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. 2) affects the dynamic characteristics of the tower. Hence, the upper instrumented sections (Fig. 7) were at the crowning (about 2.0 m from the top and 52.0 m from the ground level) 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 31/07/2012 and from 9:00 of 01/08/2012 to 6:00 of 01/08/2012. One typical sample of acceleration time series recorded at the upper instrumented level is illustrated in Fig. 8: 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 sampling frequency was 200 Hz, which is much higher than that required for the investigated structure, as the signif-icant 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 144,000, with a sampling interval of 0.025 s.

The extraction of modal parameters from ambient vibration data was carried out by using the Frequency Domain Decomposition (FDD) technique [20] in the frequency domain and the data-driven Stochastic Subspace Identification (SSI) method [21,22], in the time domain; these techniques are available in the commercial software ARTeMIS [23]. More specifically, the FDD was mainly applied on site to quickly estimate the dynamic characteristics of the structure, whereas back in the office those estimates were refined using the SSI.

The modal identification was performed using time series of 3600 s (corresponding to more than 3000 times the fundamental period of the tower), so that the well-known condition (see e.g. [24]) about the length of the acquired time window (that should not be less than 1000–2000 times the structure's fundamental period) is largely satisfied. It is worth mentioning that an appropriate length of the acquired time window [24] is mandatory to obtain reliable and accurate modal identification from ambient vibration data; in fact, all the output-only identification techniques (such as FDD and SSI) assume that the excitation input is a zero-mean Gaussian white noise and this assumption is as closely verified as the length of the acquired time window is longer.

Since past experimental investigations of similar structures [6,11,13] highlighted clear effects of temperature on the natu-ral frequencies, a second acquisition system was used during the dynamic tests to measure the temperature in three differ-ent points of the tower: both indoor and outdoor temperature were measured on the S-W load-bearing wall, whereas only the outdoor temperature was measured on the S-E front. It is worth mentioning that the changes of outdoor temperature on the struc-ture were very significant and ranged between 25 °C and 55 °C, whereas slight variations were measured by the indoor sensor (29–30 °C) due to the high thermal inertia of the load-bearing walls.

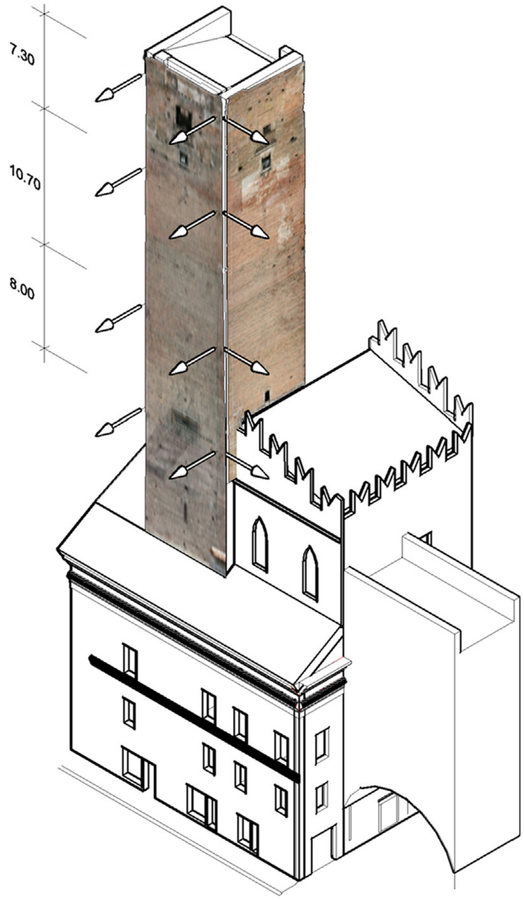

Fig. 7. Sensor layout adopted in ambient vibration testing of the tower (dimensions in m).

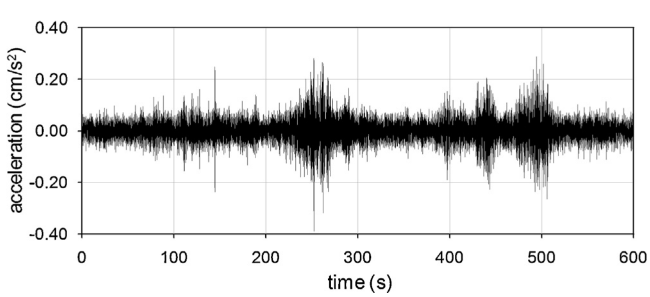

Fig. 8. Sample of acceleration recorded at the top of the tower.

4.2. Dynamic characteristics of the tower

Notwithstanding the very low level of ambient excitation (Fig. 8) 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 Fig. 9 and refer to the acceleration data recorded in the time window 16:00–17:00 of 31/07/2013. The inspection of the mode shapes (Fig. 9) allows the following comments on the dynamic characteristics of the tower:

- 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. 9a) is dominant bending in the N-E/S-W plane whereas the modal deflections of B2 (Fig. 9b) belong to the orthogonal N-W/S-E plane;
- the third mode B3 (Fig. 9c) involves bending in the N-E/S-W plane, with slight (but not negligible) components in the orthogonal plane;
- only one torsion mode (T) was identified (Fig. 9d) and the corresponding natural frequency was 4.77 Hz (in the examined time window);
- the last identified mode is local (L) and only involves deflec-tions of the upper portion of the tower (Fig. 9e). The mode shape 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 discon-tinuities) observed in the upper part of the tower during the preliminary visual inspection.

It is worth noting that the fundamental frequencies of about 1 Hz exactly fall in the range of dominant frequencies characterizing the earthquakes recorded in Mantua on May 2012 and, more generally, the earthquakes expected in the Mantua region. The frequency of fundamental modes and the presence of one local mode, involving the top of the tower, explain the fall of small masonry pieces from the upper part of the building, reported during the earthquake of 29/05/2012. Hence, both visual inspection and OMA confirm the concerns about the seismic vulnerability of the building.

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

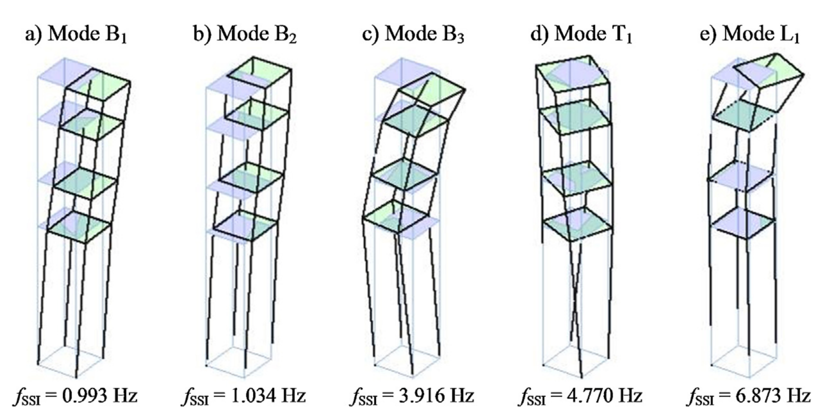

Fig. 9. Sample of vibration modes generally identified from ambient vibration measurements (SSI, 31/07/2012, 16:00–17:00).

Table 1
Statistics of the natural frequencies identified (SSI) from 31/07/2012 to 02/08/2012.

Mode	f_{ave} (Hz)	σ_f (Hz)	f_{min} (Hz)	f_{max} (Hz)
1 (B_1)	0.981	0.018	0.957	1.014
2 (B_2)	1.026	0.011	1.006	1.052
3 (B_3)	3.891	0.025	3.857	3.936
4 (T_1)	4.763	0.022	4.714	4.802
5 (L_1)	6.925	0.037	6.849	6.987

B: bending mode; T: torsion mode; L: local mode.

through the mean value (f_{ave}), the standard deviation (σ_f) and the extreme values (f_{min}, f_{max}) 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).

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 modal frequencies and temperature, the simplest approach is to plot estimates of each frequency versus temperature, as it is illustrated in Fig. 10. More specifically, Fig. 10 shows the natural

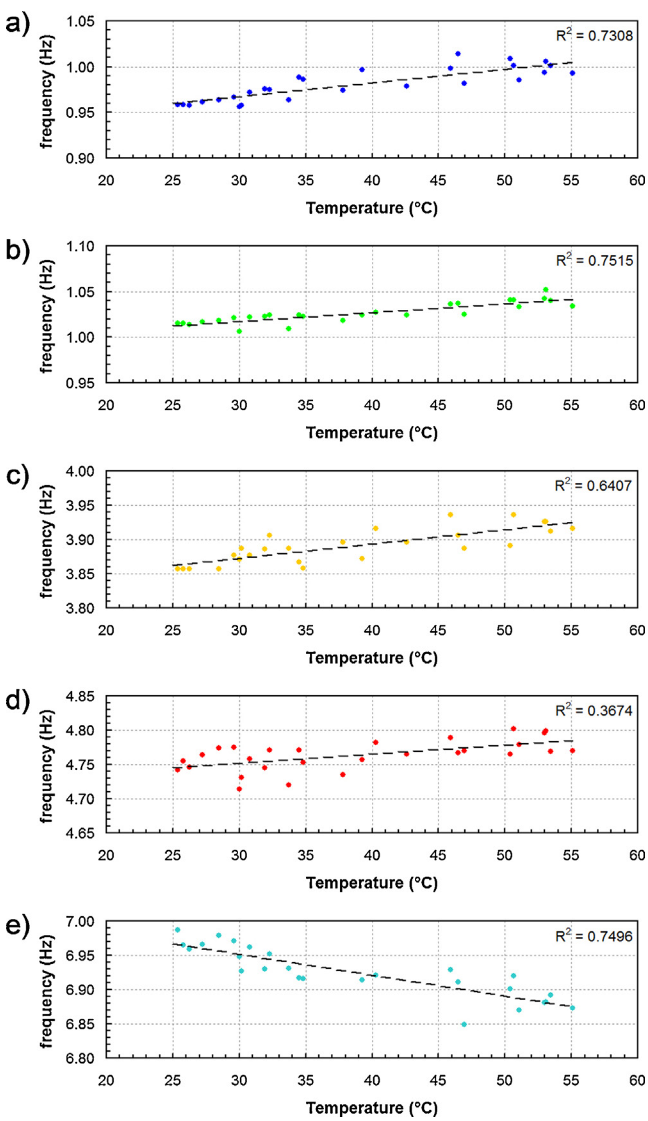

Fig. 10. Frequency (SSI) variation versus measured outdoor temperature (S-W front). a: mode B1; b: mode B2; c: mode B3; d: mode T1; e: mode L1.

frequencies, identified via SSI, versus the hourly averaged temperature, measured outdoor on the S-W front; each graph also shows the best-fit line (which has been added as a visualisation aid) and the coefficient of determination R^2 [25].

Fig. 10a–c shows that the natural frequencies of bending modes B1–B3 increase with increased temperature and exhibit an almost linear correlation with the temperature, with R^2 ranging between 0.64 (mode B3, Fig. 10c) and 0.75 (mode B2, Fig. 10b). The frequency–temperature correlation seems less strong for the torsion mode (Fig. 10d) 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 (Fig. 10e) decreases as the temperature increases and the coeffi-cient of determination assume a high value ($R^2 \cong 0.75$). This result suggests that the thermal expansion of materials in a very hetero-geneous area of the structure causes a general worsening of the connection between the masonry portions; hence (and again in agreement with the main observation of the visual inspection), fur-ther evidence is provided of the poor state of preservation of the upper part of the tower.

5. Conclusions

The paper demonstrates the importance of a multidisciplinary approach in the assessment of historic buildings and especially in prompt post-earthquake diagnostic investigation. Integrating the information obtained from visual inspection, on-site survey and ambient vibration tests turned out to provide a knowledge of utmost importance and clear indication of the possible building evolution, masonry changes, local and global state of preservation; in addition, the positions that are more suitable to further ND or MD tests and material sampling were clearly identified.

Visual inspection of all main bearing walls firstly revealed 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 of the tower; the corresponding mode shapes is dominant bending, with significant components along the two main planes of the tower. It is worth mentioning 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 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 with increased temperature, suggesting that the thermal expansion of materials in a very heterogeneous area of the structure causes a general decrease of the connection between the masonry portions.

These results clearly highlighted the critical situation of the upper part of the tower, pointing out the need for structural interventions to be carried out. While waiting the retrofit design, a simple dynamic monitoring system (including three highly sen-sitive accelerometers and one temperature sensor) was installed in the tower, with structural health monitoring and seismic early warning purposes [26]. The main objectives of the permanent mon-itoring are: (a) evaluating the dynamic response of the tower to the expected sequence of far-field earthquakes; (b) evaluating the effects of temperature on the natural frequencies of the tower; (c) detecting any possible anomaly or change in the structural behaviour by identifying the frequency shifts that are not associated to "normal" temperature changes and (d) evaluating the effects of the future repair intervention.

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