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ON-SITE INVESTIGATION AND DYNAMIC MONITORING FOR THE POST-EARTHQUAKE ASSESSMENT OF A MASONRY TOWER

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Abstract. *On May 29th, 2012 a strong earthquake occurred in the region Emilia-Romagna but it was significantly felt also in Lombardy and damages were reported on several Cultural Heritage buildings placed in the city of Mantua. Therefore, the VIBLAB (Laboratory of Vibration and Dynamic Monitoring of Structures) of Politecnico di Milano was committed to assess the structural condition and the seismic behavior of the "Gabbia" tower, which is the tallest historic tower in Mantua. After a brief description of the masonry tower, about 52.0 m high and dating back to the XIII century, the paper focuses on the tasks involved to assess the structural safety:(a) historic and documentary research;(b) geometric survey and on-site survey of the crack pattern and structural discontinuities;(c) non-destructive and slightly destructive tests of materials on site;(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. Visual inspection of all main 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 local modes involving the upper part of the tower were clearly identified by applying different output-only techniques to the ambient response data collected for more than 24 hours. Furthermore, the continuous dynamic monitoring system, installed on the tower in December 2012, and the software developed for processing the collected data is described. Typical results of the dynamic monitoring are presented and discussed, along with the analysis of some earthquake records collected since the beginning of monitoring.*

1 INTRODUCTION

Ambient vibration testing (AVT) and continuous monitoring of the structural response under ambient excitation are well-known non-destructive methodologies, generally aimed at identifying the dynamic characteristics of a structure from output-only records using operational modal analysis (OMA) techniques. Although AVT has become the primary modal testing method of civil engineering structures, its application to historic structures can be considered a rather recent topic and only a few complete case studies are reported in the literature [1]-[4]. On the other hand, there is an increasing interest on this topic since the preservation of Cultural Heritage is of primary concern in many countries all over the world. In addition, AVT is especially suitable to historic structures because it is a fully non destructive and sustainable way of testing that does not interfere with the normal use and does not involve additional loads rather than those due to normal operational conditions. Often AVTs have been performed to investigate the dynamic behaviour of historic towers [1]-[3] and minarets [4] because these structures, that are usually slender and subjected to significant dead loads, might exhibit high sensitivity to dynamic actions, such as traffic-induced micro-tremors, swinging of bells, wind and earthquakes. In addition, the cantilever-like behaviour of towers suggests the use of continuous monitoring systems (consisting of few sensors placed in the upper part of the structure), with preventive conservation and/or structural health monitoring purposes [5].

The paper presents the results of a continuous dynamic monitoring program carried out on the tallest historic tower in Mantua, Italy, after the seismic sequence of Spring 2012. On May 29th, 2012 a strong earthquake occurred in the region Emilia-Romagna but it was significantly felt also in Lombardy and damages were reported on several Cultural Heritage buildings placed in the city of Mantua, where Politecnico di Milano has a large campus. Therefore, the VibLab (Laboratory of Vibration and Dynamic Monitoring of Structures) of Politecnico di Milano was committed to assess the structural condition and the seismic behavior of the "Gabbia" tower, which is the tallest historic tower in Mantua. The Spring 2012 earthquake highlighted the high vulnerability of the historic architectures particularly in the South part of the province of Mantua and in the neighbouring Emilia-Romagna region where several brittle collapses of towers, fortification walls and castles occurred, despite the supposed low seismic risk of the area. The tower (Fig. 1), about 54.0 m high and surrounded by an important historic building, is one of the symbols 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], [5]: (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 (i.e. sonic pulse velocity tests and flat-jack tests); (e) dynamic tests in operational conditions; (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 [6].

Visual inspection of all main 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 local modes involving the upper part of the tower were clearly identified by applying different output-only techniques to the ambient response data collected for more than 24 hours on the historic structure. 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. With this motivation, and in order to allow better indoor inspection of the tower bearing walls, a metal scaffolding and a light wooden roof was installed inside the tower. Hence, a second dynamic test was performed – aimed at checking the possible effects of scaffolding and wooden roof on the modal characteristics of the structure – and a simple permanent dynamic monitoring system was installed in the tower, with seismic and structural health monitoring purposes. The instrumentation installed inside the tower consists of a 4-channel data acquisition board, with 3 piezoelectric accelerometers and 1 temperature sensor. A binary file, containing 3 acceleration time series (sampled at 200 Hz) and the temperature data, is created every hour, stored in an industrial PC on site and transmitted to Politecnico di Milano for being processed.

The main objectives of the long-term monitoring 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 building; (c) detecting any possible anomaly or change in the structure behavior and (d) evaluating the effects of the future strengthening intervention.

After a brief description of the tower and the experimental investigation, the paper focuses on the continuous dynamic monitoring system (installed on the tower in December 2012) and the software developed for processing the collected data. Subsequently, typical results of the dynamic monitoring are presented and discussed, along with the analysis of few earthquake records collected since the beginning of monitoring.

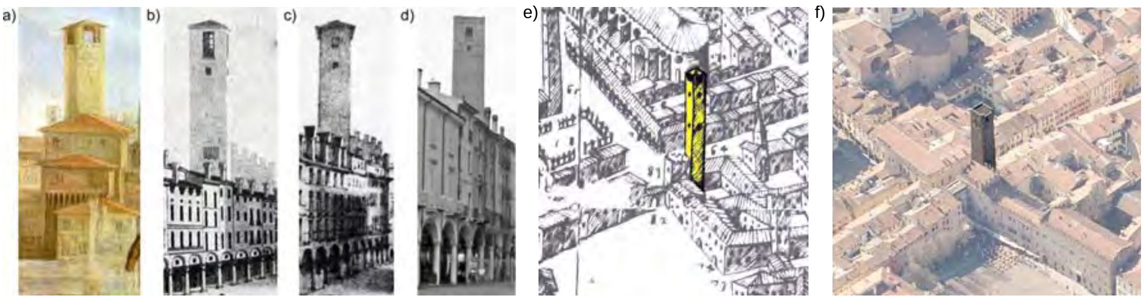

Figure 1: The Gabbia tower and the surroundings: XVII (a), XVIII (b), XIX (c) and XX (d) century (e) view of the XVII century [8] (f) recent view from East.

2 THE TOWER: GENERAL DESCRIPTION

The Gabbia Tower is the tallest tower in Mantua, overlooking the historic centre listed within the UNESCO Heritage (Figs. 1 and 2). The tower has been of private ownership for long time and only during the eighties, it passed to the Mantua Municipality. Few historic documents are available on the tower and its evolution [7]. 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, governing 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. 2b) and the access to the lower portion and to the base of the building is not possible. The Tower was used in the XVI century as open-air jail, hosting a hanged dock, a cage (“gabbia” in Italian), on the S-W front (Fig. 2). The tower, about 54.0 m high, is built in solid masonry bricks and has an almost squared plan; the load bearing walls are about 2.4 m thick up to the upper levels (Fig. 2) where the thickness of the masonry cross-section decreases to about 0.7 m. The top part of the building has a two level lodge, which hosted in XIX century the observation and telegraph post. A wooden staircase reached the lodge but it is no more practicable since several years due to the lack of maintenance. The inner access to the tower was re-established recently (October 2012) through provisional scaffoldings.

The original layout of the surrounding structures is unknown. At present the tower is part of an important palace, evolved since the XIII century [7], complicating the geometry of the structure and the mutual links. In general, the load bearing walls of the palace are not effectively linked but just drawn to the tower's masonry walls. Few historic documents are available on the past interventions on the tower but the observation of the masonry reveals passing-through discontinuities in the upper region, that are conceivably related to the tower evolution (Figs. 3 and 4). Traces of past structures are visible on all fronts (Fig. 2) and the presence of merlon-shaped discontinuities (Fig. 3b-d) suggests modifications and further adding at the top of the tower, reconstructed by the stratigraphic principles. 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. 4a,b); in the same region concentrated changes of the masonry texture reveal local repair (Fig. 4c-4e).

A first hypothesis, based on the surface discontinuity survey, could recognise six main building phases (Fig. 3a): (i) erection of the main building up to about 46.0 m (probably concluded in 1227) and recognisable by the surface workmanship of the scratched bricks (Fig. 4a,b); (ii) subsequent addition, up to the crenellation level (Fig. 3b-d); (iii) adding of 4 corner piers supporting a four side roof; (iv) opening infilling and construction of the windows, crowning and the new roof; (v); repair of the South corner (Fig. 4c-e).

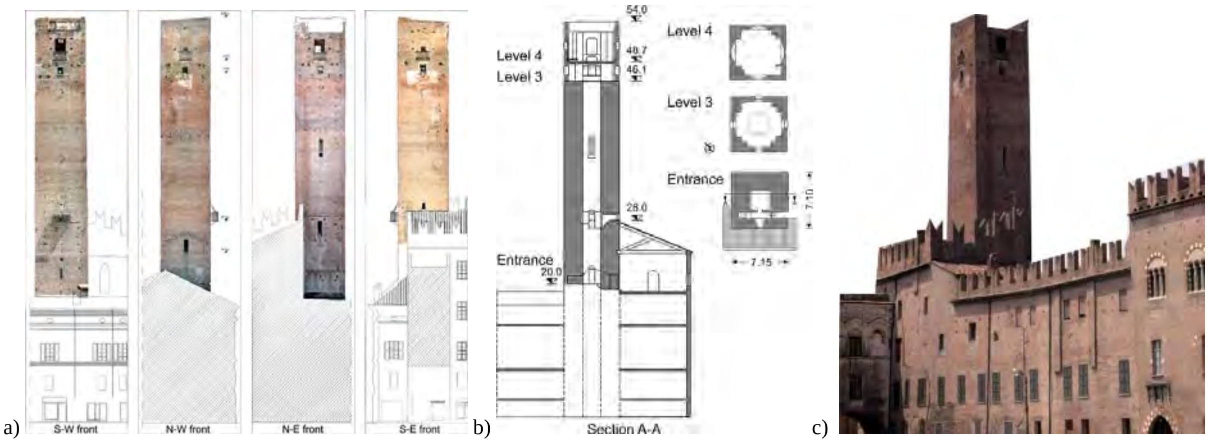

Figure 2: Fronts (a) , section (b) and view from East (c) of the Gabbia Tower.

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 using a movable platform and, once the inner access was re-established through a metal scaffolding, the inner fronts were inspected as well.

This preliminary survey was 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, 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.

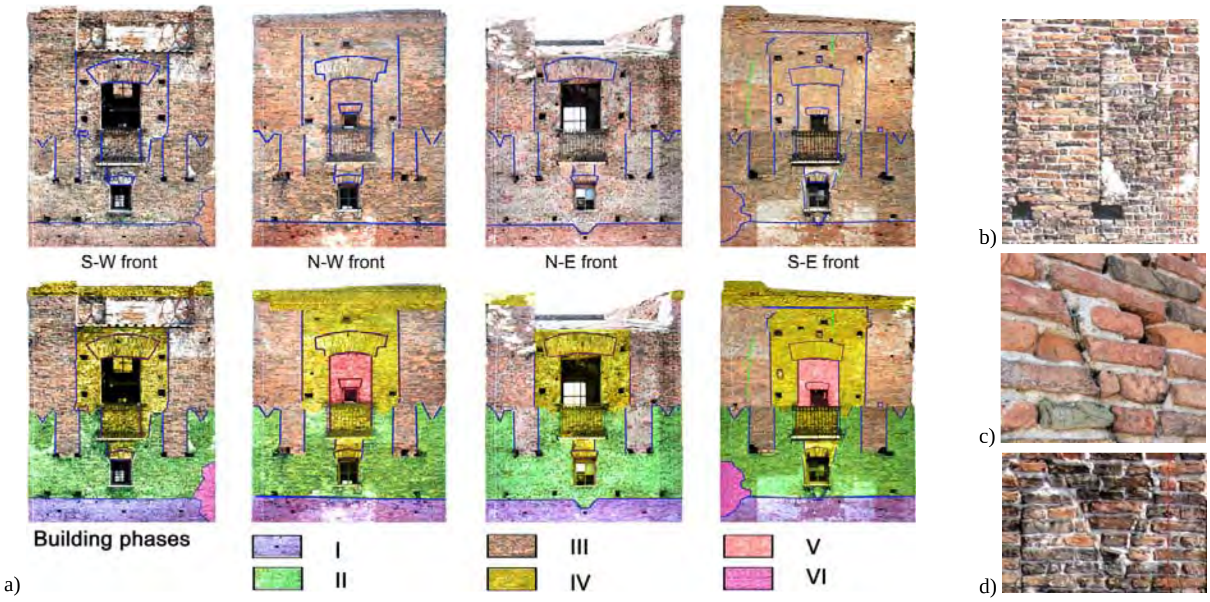

Figure 3: Map of the structural discontinuities and of the supposed building phases (a) and details of the probable merlons embedded in the masonry texture (b-d).

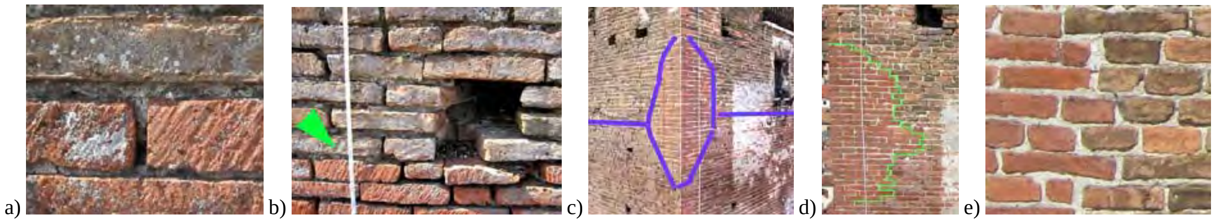

Figure 4: Change of the surface workmanship at about 8.0 m from the top (the bricks of the lower portion are superficially scratched) (a,b) and probable repairs revealed by local changes of the masonry texture (c-e)

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) and to the upper 8 m of the tower, respectively. Excepting the upper part of the tower (i.e. a portion about 8.0 m high, Figs. 2 and 3), visual inspection did not reveal evident structural damage but only superficial decay of the materials (mainly mortar joint erosion, 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, double flat jacks and laboratory tests on sampled mortars and bricks confirm the soundness and the compactness of the masonry until the height of about 46.0 m. 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 revealed that the Young's modulus is larger than 3.00 GPa. Similar information result from the laboratory tests on sampled bricks and mortars. Thermovision inspections applied on several walls of the tower portion embedded in the surrounding palace detected local niches and openings infilled even recently.

On the contrary, in the upper 8.0 m of the tower (Fig. 2 and 4) significant damages were observed, related to the abovementioned detachment of the several construction phases and worsened by the natural decay. More specifically, 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. 5). Critical areas are the infillings between the merlons (Figs. 3 and 5), 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. 5) so that the prevention of local instability of such masonry portions should be one of the intervention priority on the tower. Moreover, the collected information by the visual inspection allowed to draw up the stratigraphic survey of the fronts. The stratigraphic survey and the accurate mapping of the structural discontinuities allowed to support some concerns on the seismic behaviour of such portions. Because of the lack of effective links except the friction, some unrestrained portions could overturn for the twinned actions of the earthquake and of the roof thrust. Low intensity actions, like after-shocks or far-field earthquakes could slightly move such weakly restrained elements decreasing the adhesion and accentuating the boundaries.

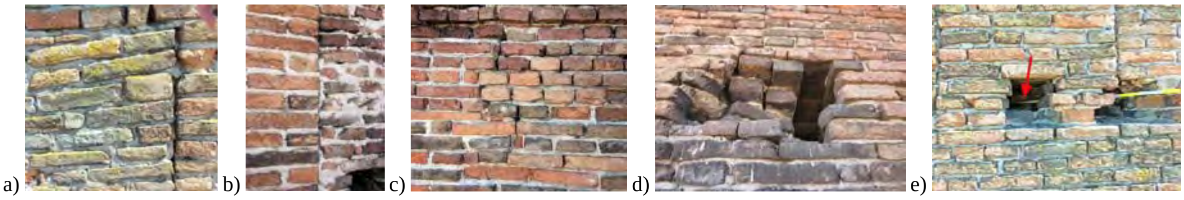

Figure 5: Details of the typical discontinuities on top of the tower (a,b), of the lack of horizontality of the masonry courses (d) and of the infillings between the supposed merlons on the N-W front

Fig. 6 shows an example of the detailed survey of the masonry textures aimed at recognising the structural discontinuities and the boundaries of the weakly restrained portions. Based on this investigation, the first evaluation of out-of-plane seismic behaviour for each recognised masonry portion not effectively linked was carried out (Fig. 6c). This procedure, implemented according the most recent technical literature and the Italian seismic code, gives an overview of the seismic vulnerability related to the building transformation over time and the effect of local damage. 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. 2), 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. In fact, 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.

4 PRELIMINARY DYNAMIC TESTS

Two ambient vibration tests (AVTs) were conducted on the tower: between 31/07/2012 and 02/08/2012, and on 27/11/2012. The second test was performed after the installation of a metallic scaffolding and a wooden roof, to check the possible effects of those additions on the dynamic characteristics of the structure. Fig. 7a shows the sensor layout adopted in the tests; the same cross-sections were instrumented even if in the first AVT, since the inner access to tower was not available in Summer 2012, the sensors were mounted on the outer side of the walls. In both AVTs, the excitation was provided only by wind and micro-tremors and very low level of ambient excitation existed during the tests, with the maximum recorded acceleration being constantly lower than 0.4 cm/s^2. This required high sensibility sensors and accurate preliminary calibration. Typical results in terms of natural frequencies and mode shapes are shown in Fig. 4b and allow the following comments:

1. two closely spaced modes were identified around 1.0 Hz. These modes (Fig. 7b) are dominant bending (B) and involve flexure in the two main planes of the almost squared tower,

respectively;
2. the third mode (Fig. 7b) involves dominant bending in the N-E/S-W plane with slight components also in the orthogonal N-W/S-E plane;
3. just one torsion mode (T) was identified (Fig. 7b);
4. the last identified mode is local (L) and only involves deflections of the upper portion of the tower (Fig. 7b). The mode shape looks dominant bending, with significant components along the two main planes of the structure. The presence of this local vibration mode provides further evidence of the structural effect of the change in the masonry quality and morphology observed on top of the tower during the visual inspection. On the other hand, both visual inspection and operational modal analysis confirm the concerns about the seismic vulnerability of the building and explain the fall of small masonry pieces from the upper part of the tower, reported during the earthquake of May 29th 2012.

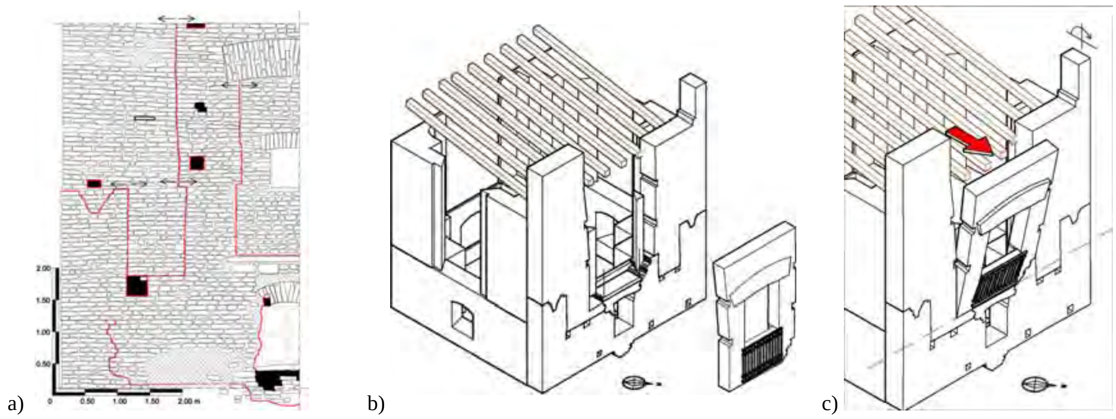

Figure 6: (a) Map of the structural discontinuities; (b) identification of the weakly restrained masonry portions and (c) of the possible overturning mechanisms due to seismic actions.

The results of the second AVT, in terms of identified natural frequencies and mode shapes, are shown in Fig. 7c and can be summarised as follows:
1. beyond the difference in terms of natural frequency (that are conceivably related to the temperature effects [9], the mode shapes of bending modes B1-B3 did not exhibit significant changes (Fig. 7b,c). Hence, the metallic scaffolding and the wooden roof practically do not affect those modes;
2. on the contrary, the mode shape T1 (Fig. 7c) now involves both torsion and bending. The identified frequency did not change appreciably with respect to the first dynamic survey, but the mode shape looks significantly different. The torsion component is still dominant in the lower portion of the structure, while the upper part is characterized by dominant bending with significant components along the two main planes of the tower. In other words, after the installation of the wooden roof the deformed shape of mode T1 (Fig. 7c) becomes a sort of superposition of previous modes T1 (Fig. 7b, lower part of the structure) and L1 (Fig. 7b, upper part of the structure). Furthermore, the previous mode L1 was no more detected;
3. the previous local mode L1 (Fig. 7b) has been "replaced" by another local mode, with higher frequency of 9.89 Hz and involving torsion of the upper part of the tower (Fig. 7c). This local mode, not identified in the first AVT, is probably related to the increase of connection between the masonry walls of the upper part of the tower induced by the new covering.

As a further comment, it seems that especially the wooden roof, even if very light, affects the dynamic characteristics of the upper part of the building. The effect is two-fold: on one hand, the roof acts as a mass added in a vulnerable area (so that a possible decrease of the natural frequency of previous local mode is generated) whereas, on the other hand, the roof has binding effect on the dismantled masonry characterizing the upper part of the tower. It is further noticed that differences in terms of natural frequency were also detected between the two tests, although conceivably related to temperature effects.

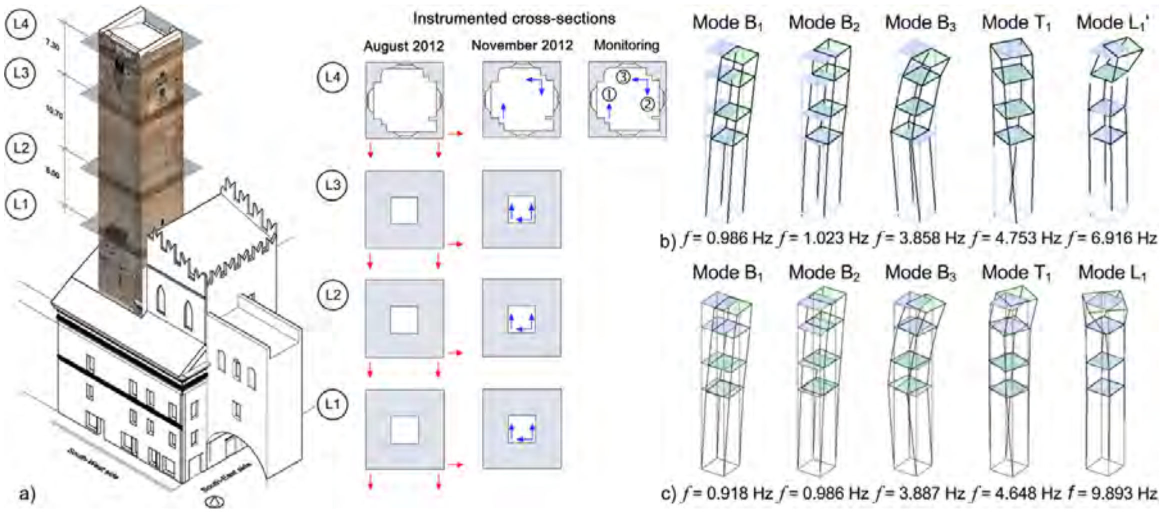

Figure 7: (a) Instrumented cross-sections and sensors layout during the dynamic tests performed on Summer 2012, November 2012 and the dynamic monitoring; (b) vibration modes generally identified during the first AVT (SSI-Data, 31/07/2012, 21:00-22:00) and (c) in the second AVT (SSI-Data, 27/11/2012).

5 PERMANENT DYNAMIC MONITORING AND DATA ANALYSIS

Few weeks after the execution of the second AVT, a simple dynamic monitoring system was installed in the tower. The system is composed by a 4-channel data acquisition system (24-bit resolution, 102 dB dynamic range and anti-aliasing filters) with 3 piezoelectric accelerometers (WR model 731A, 10 V/g sensitivity and ±0.50 g peak). Furthermore, a temperature sensor is installed on the S-W front, measuring the outdoor temperature. The digitized data are transmitted to an industrial PC on site. A binary file, containing 3 acceleration time series (sampled at 200 Hz) and the temperature data, is created every hour, stored on the local PC and transmitted to Politecnico di Milano for the being processed.

The response of the tower is measured in 3 points, belonging to the cross-section at the crowning level of the tower. During the monitoring, data sets related to seismic episodes were recorded, as well. The continuous dynamic monitoring system has been active since December 2012. The data files received from the monitoring system are managed by a software developed in LabVIEW [11] and including preliminary statistical analysis of data, preprocessing and automatic recognition and extraction of possible seismic events. The results of modal identification herein presented were obtained applying the SSI-data method available in the commercial software ARTeMIS [11], [12].

During the examined time period, the monitoring system acquired the tower's response to different earthquakes occurred in the neighbouring regions. Fig. 8 shows examples of the accelerations recorded during some seismic events, with the maximum response exceeding several times the highest level of normally observed ambient vibrations (0.4-0.5 cm/s^2). It should be noticed that the strongest seismic event (Fig. 8c, corresponding to the earthquake which hit the Garfagnana area in Tuscany on 21/06/2013) produced significant effects on the *Gabbia Tower*, as it will be discussed in the last part of this section.

Fig. 9a presents the evolution of the outdoor temperature on the S-W front during the period from 17/12/2012 to 13/08/2013 and shows that the temperature changed between –2°C and +45°C with significant daily variations in sunny days. Automated identification of the modal frequencies from the datasets collected in the same period provided the frequency tracking shown in Fig. 9b. The inspection of Fig. 9 firstly suggests that the slight fluctuation of the natural frequencies of global modes (B_1-B_3 and T_1, Figs 7b,c) follows the temperature variation. In order to better demonstrate the effect of changing temperature on the fluctuations of the modal frequencies, the simplest approach is to plot each frequency with respect to temperature. For example, Fig. 10a,b shows the natural frequencies of modes B_1 and B_2 plotted with respect to temperature along with best fit lines. The plots in Fig. 10a,b, referring to the period from 07/01/2013 to 14/06/2013, reveal a clear dependence of the investigated natural frequencies on temperature and confirm what already observed in the first dynamic survey [9]: the natural frequency of global modes tends to increase with increased temperature. This behaviour, observed also in past experiences on masonry towers [5] can be explained through the closure of superficial cracks, minor 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.

The time evolution of the natural frequency of the upper mode, i.e. the local mode L_1 (Figs 7b,c), deserves some concern because the trend of this frequency is very different from the others (Fig. 10): the modal frequency exhibits more significant fluctuations and clearly decreases in time, from an initial value of about 10.0 Hz to a final value of about 8.5 Hz at the end of the examined time period.

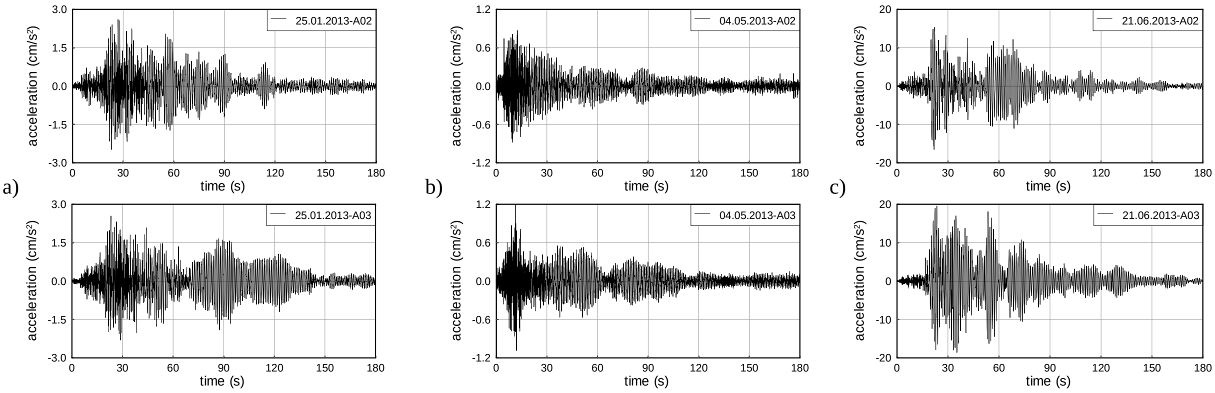

Figure 8: Seismic responses recorded on the tower: (a) 25/01/2013, (b) 04/05/2013 and (c) 21/06/2013.

A careful inspection of Fig. 9 reveals that 3 clear drops of the natural frequency took place: (a) between 03/02/2013 and 04/02/2013; (b) between 14/03/2013 and 15/03/2013 and (c) between 13/04/2013 and 15/04/2013. These drops divide the analyzed time period in 4 parts, that are also easily observable by plotting the modal frequency versus the measured outdoor temperature, as shown in Fig. 10. Fig. 10 highlights that the clouds of temperature-frequency points, corresponding to each of the 4 different periods, are characterized by similar slope of the best fit line, whereas the average frequency value significantly decreases. The observed behaviour suggests the progress of a possible damage mechanism, conceivably related to the effect exerted by the wooden roof with increased temperature, and confirms – once more – the poor structural condition and the high seismic vulnerability of the upper part of the tower, highlighting the urgent need for preservation actions to be carried out. As previously pointed out, data sets related to far-field seismic events were recorded on 25/01/2013, 04/05/2013, 19/06/2013 and 21/06/2013. The latter event, corresponding to a significant earthquake occurred in the Garfagnana region, determined acceleration responses (Fig. 8c) on the top of the tower, that were more than 40 times larger than the usual ambient vibration responses

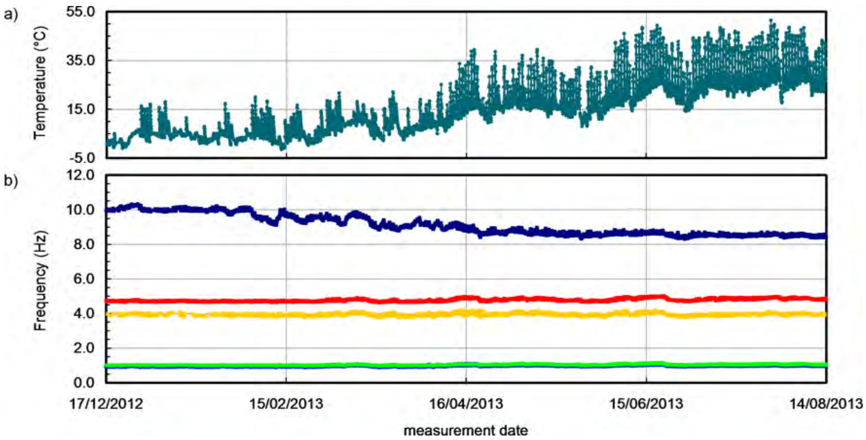

Figure 9: Time evolution of: (a) the outdoor temperature measured on the S-W front; (b) the natural frequencies identified with the SSI-data method.

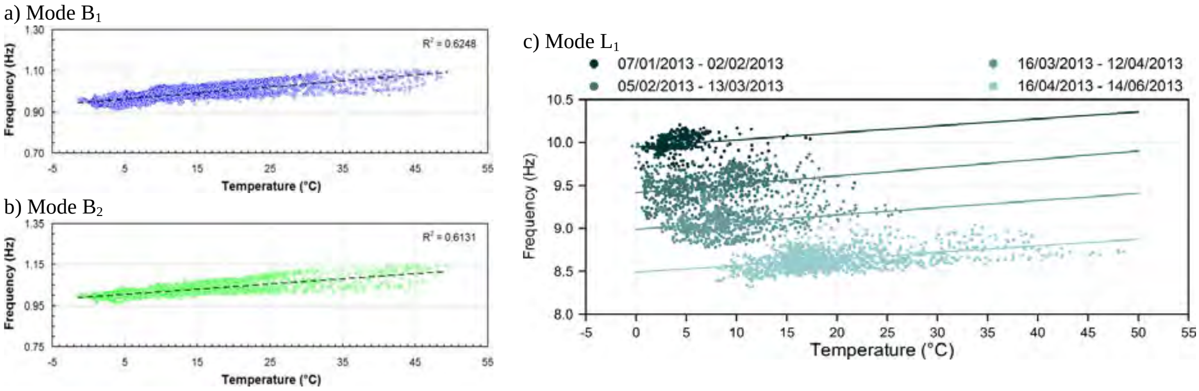

Figure 10: Natural frequency of modes B1 (a), (b) and L1 plotted with respect to the outdoor temperature in the period between 07/01/2013 and 14/06/2013

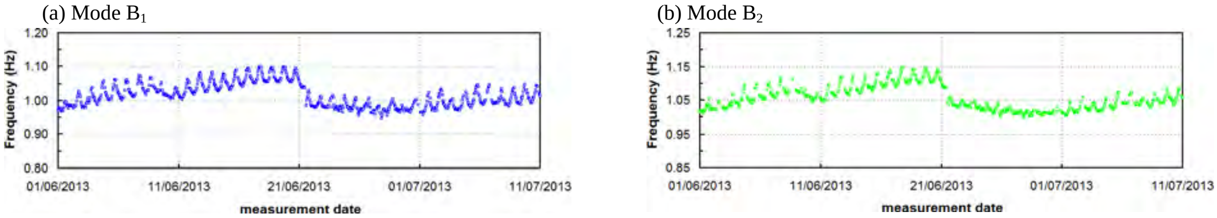

Figure 11: Zoom of the natural frequency of modes B1 (a) and B2 (b) in the period between 01/06/2013 and 10/07/2013.

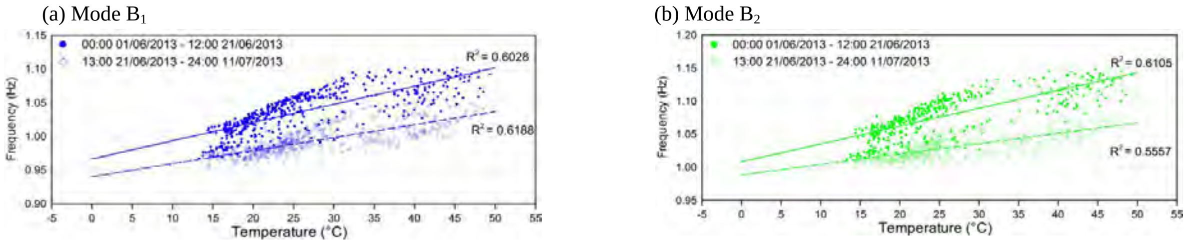

Figure 12: Change in the frequency-temperature correlation induced by the seismic event of 21/06/2013: (a) Mode B1; (b) Mode B2.

Zooming the time evolution of the natural frequencies of lower modes B_1 and B_2, as shown in Fig. 11, highlights a clear drop of those modal frequencies, corresponding to the occurrence of the seismic event on 21/06/2013. The frequency shift is even more clear by inspecting the frequency content of the data recorded in the hour before and after the seismic event, respectively. In order to investigate whether the frequency shifts are permanent or not, the frequency-temperature relationship were inspected, including data collected in the 3 weeks before and after the earthquake. The results of this check are summarized in Fig. 12, where best fit lines have been added as a visualization aid: the regression lines exhibit a remarkable variation after the seismic event, with the range of temperature variation being almost unchanged. Both the variation of the regression lines and the arrangement of the frequency-temperature points before and after the earthquake seem to indicate that the observed slight frequency shift is non reversible.

The monitoring is still ongoing, providing information about the structure behavior on a wider time window; the aim is to collect data in order to distinguish pathologic anomalies from recursive structural behavior for the drafting of eventual alarm protocols able to detect in early warning phase eventual problems and plan suitable strategies.

6 CONCLUSIONS

The paper focuses on the post-earthquake assessment of a historic masonry tower and summarizes the results of visual inspection, ambient vibration tests and long-term dynamic monitoring of the building. Visual inspection of all main 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 the SSI-data technique to the ambient response data collected for more than 24 hours on the historic structure. Furthermore, the natural frequency of this local mode tends to decrease as temperature increases, suggesting that the thermal expansion of materials in a very inhomogeneous area of the tower, causes a general decrease of the connection between the masonry portions.

These results highlighted the need for preservation actions to be carried out. For this reason, a light wooden roof and a metallic scaffolding were installed in the tower. To check the effect of both wooden roof and scaffolding on the dynamic behavior of the building, a second dynamic survey was carried out. The comparison between normal modes identified in the two dynamic tests provided the following evidences:

1. the mode shape of the lower normal modes is practically unchanged;
2. the first torsion mode changed its mode shape in the upper part of the building. More specifically, the torsion component is dominant in the lower part of the tower, whereas a local bending component prevails in the upper region;
3. a new local mode was identified, involving torsion of the upper part of the tower.

Few weeks after the second dynamic survey, a simple dynamic monitoring system was installed at the crowning level of the tower. The time evolution of the modal frequencies identified from 07/01/2013 to 06/05/2013 clearly highlights:

1. the impact of temperature on the natural frequencies of global modes (i.e. the modal frequencies increase with increased temperature);
2. the quick progress of a damage mechanism, involving the upper part of the tower, and clearly identified through the remarkable fluctuations and the significant decrease (about 15% in 4 months) of the natural frequency corresponding to a local mode;
3. the decrease of the natural frequencies of the fundamental modes detected after the occurrence of a far-field seismic event, demonstrated by the comparison of the modal peaks identified before and after the earthquake, and confirmed by the subsequent frequency tracking.

In this context, the importance of a permanent monitoring of the building appears clear; the monitoring points out in real time the eventual evolution of the state of preservation of the building, and it collects the needed information to plan possible scenarios of use and strategies of maintenance. Continuous dynamic monitoring effectively allows to check the evolution of the global behavior, the thermal effects, the occurrence of possible damage as well as the effects of exceptional events like strong winds, storms and earthquakes. As the monitoring time period extends, further information could effectively calibrate alarm protocols in early warning phase and plan suitable strategies.

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