DYNAMIC AND SEISMIC HEALTH MONITORING OF A HISTORIC MASONRY TOWER

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SUMMARY – The paper presents some results of the continuous dynamic/seismic monitoring program carried out on the tallest historic tower in Mantua, Italy. This project follows an extensive diagnostic investigation aimed at assessing the structural condition of the tower after the Italian earthquakes of May 2012.
A simple dynamic monitoring system was installed in the tower to evaluate the dynamic response especially to the expected sequence of far-field earthquakes and to check the possible evolution of the natural frequencies; the response to ambient excitation has been continuously collected in 1-hour records since late December 2012.
The paper summarizes the results of the continuous dynamic monitoring for a period of 8 months, highlighting the effect of temperature on automatically identified natural frequencies, the dynamic response to few seismic events and the key role of permanent dynamic monitoring in the diagnosis of the investigated historic building.

Key words: automated modal identification, continuous dynamic monitoring, masonry tower, seismic assessment, structural health monitoring

1. **Introduction**

Ambient vibration testing (AVT) and continuous dynamic monitoring 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, see e.g. [Magalhães and Cunha, 2011].

Although AVT has become the primary modal testing method of civil engineering structures, its application to historic structures is still quite limited. On the other hand, AVT is especially suitable to historic structures because it is a fully non destructive and sustainable way of testing, that is performed by just measuring the dynamic response under ambient excitation and does not involve additional loads rather than those due to normal operational conditions. Sometimes AVTs have been performed to investigate the dynamic behaviour of historic towers and minarets [Bennati *et al.*, 2005; Ivorra and Pallares, 2006; Gentile and Saisi, 2007; Peña *et al.*, 2010; Ramos *et al.*, 2010; Gentile and Saisi, 2013; Cabboi *et al.*, 2013; Cantieni, 2014] 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 [Ivorra and Pallares, 2006; Gentile and Saisi, 2013; Cantieni, 2014], wind and earthquakes. In addition, the cantilever-like behaviour of towers suggests the use of simple dynamic monitoring systems, including few sensors installed in the upper part of the structure, with preventive conservation and/or structural health monitoring purposes [Ramos *et al.*, 2010; Cabboi *et al.*, 2013; Cantieni, 2014].

The paper presents the results of a continuous dynamic and seismic monitoring program carried out on the tallest historic tower in Mantua, Italy [Zuccoli, 1988]. This project follows an extensive diagnostic investigation [Saisi and Gentile, 2014] carried out between July and November 2012 to assess the state of preservation of the tower after the Italian earthquakes of May 2012. The first part of the research [Gentile and Saisi 2014] included: (a) historic and documentary research; (b) on-site survey and visual inspection of the load-bearing walls and structural discontinuities; (c) non-destructive and minor-destructive tests of materials on site; (d) laboratory tests on sampled materials and (e) preliminary AVTs.

The results of the post-earthquake investigation highlighted the poor structural condition and the high vulnerability of the upper part of the tower, pointing out the need for structural interventions to be carried out. Furthermore, it was decided to install a simple dynamic monitoring system in the tower, as a part of the health monitoring process helping the preservation of the historic structure.

The main objectives of the continuous dynamic 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 [Ramos *et al.*, 2010; Cabboi *et al.*, 2013; Cantieni, 2014]; (c) detecting any possible anomaly or change in the structure behavior. Furthermore, another possible long-term goal is evaluating the effects of the future strengthening intervention.

After a brief description of the investigated tower and the results of the post-earthquake assessment, full details are given in the paper on the monitoring system, the methodologies used to process the collected data, the response to far-field earthquakes and the results of the continuous dynamic monitoring for a period of 8 months.

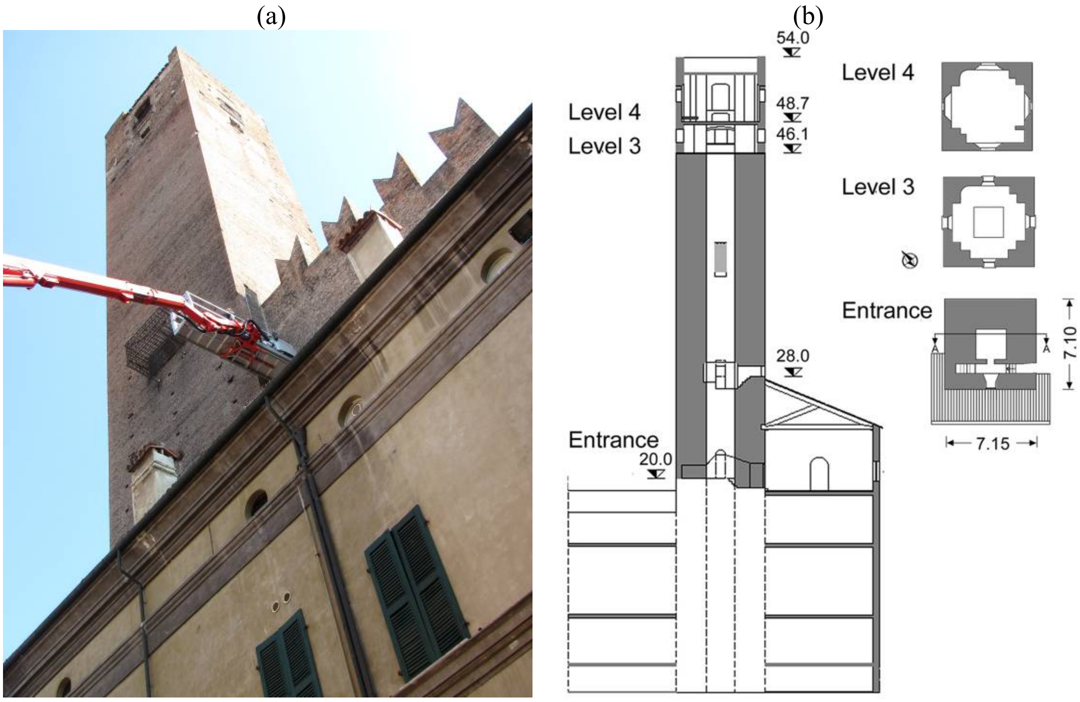


Fig. 1. *(a) View of the S-W front of the Gabbia Tower in Mantua, Italy; (b) Section of the tower (dimensions in m)*

Fig. 1. (a) Vista S-W della Torre della Gabbia in Mantova; (b) Sezione della torre (dimensioni in m)

2. **Description of the tower and state of preservation**

The *Gabbia Tower* [Zuccoli, 1988], about 54.0 m high, is the tallest tower in Mantua and is named after the hanged dock built in the XVI century on the S-W front (Fig. 1a) and originally used as open-air jail. Since the XIII century, an important palace progressively evolved around the tower, complicating the geometry of the structure and the mutual links between the walls.

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 part, hosting a two level lodge (Fig. 1b), where the corner masonry section decreases to about 0.7 m. A wooden staircase reached the lodge but it was no more in function since the 90's, 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 [Zuccoli, 1988]. Despite the construction starting 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 located at the ground level but at a higher position (Fig. 1b).

No extensive information is available on the tower past interventions but the survey of the masonry texture reveals passing-through discontinuities in the upper part of the building (Fig. 2), with those discontinuities being conceivably related to the tower evolution phases. Traces of past structures on all fronts, the presence of merlon-shaped discontinuities (Fig. 2) and the available historic pictures (Fig. 3) suggests modifications and successive adding in the upper part of the tower. A first hypothesis, based on the surface discontinuity survey, could recognize the main building phases partially illustrated in Figure 2 and 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 supporting a four side roof; iv-v) opening infilling and construction of windows, crowning and new roof; vi) repair of the South corner.

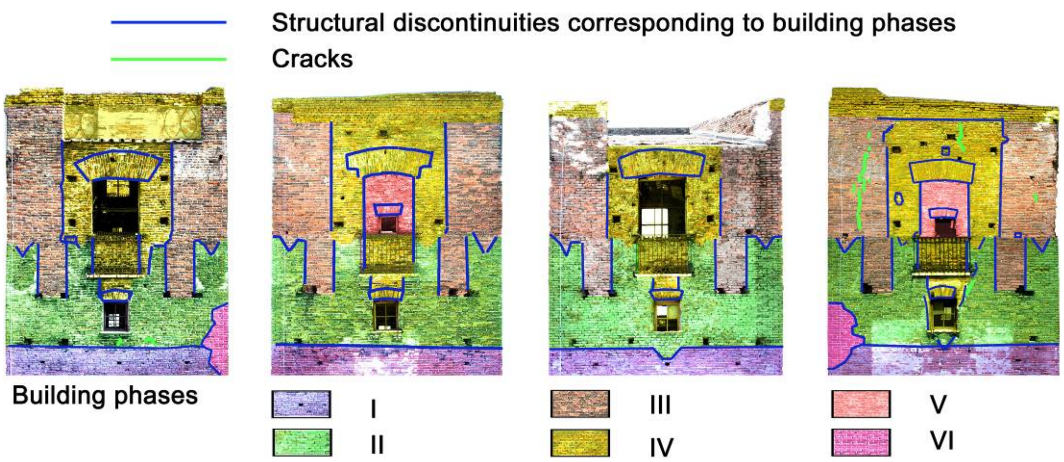


Fig. 2. *Structural discontinuities detected in the upper part of the tower and supposed building phases*

Fig. 2. Discontinuità strutturali osservate nella parte superiore della torre e fasi costruttive ipotizzate

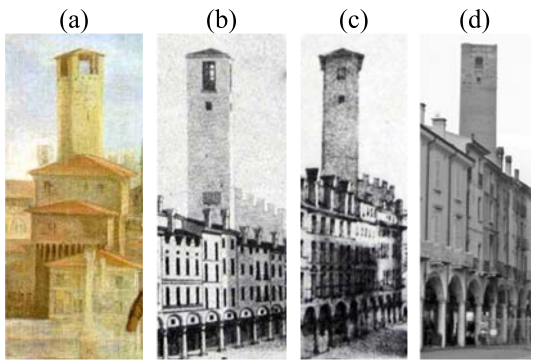


Fig. 3. *Views of the tower dating back to: (a) XVII century; (b) 1830; (c) 1852 and (d) present days*

Fig. 3. Viste della torre risalenti: (a) al XVII secolo; (b) al 1830; (c) al 1852; (d) ai nostri giorni

On-site survey of all outer fronts of the tower was performed between 30/07/2012 and 02/08/2012, using a movable platform (Fig. 1a). Survey of the inner load-bearing walls and local tests were carried out lately (November 2012), once the inner access was re-established.

The visual inspection [Gentile and Saisi, 2014] highlighted two different structural conditions, that were associated to:

(a) the main part of the building, until the height of about 46 m from the ground level;

(b) the upper region of the tower (Fig. 2).

No evident structural damage was observed in the lower part of the tower, where superficial decay of the materials was mainly detected. After the provisional scaffolding installation inside the tower, pulse sonic tests and 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 top region, about 8.0 m high) is characterized by extensive masonry decay and the presence of several discontinuities (Fig. 2) due to the historic evolution of the building (Fig. 3). The poor state of preservation of this region, characterized by lack of connection or local detachment of the construction phases, was confirmed by: (a) the low average value of the sonic velocity (600 m/s) obtained from pulse sonic tests; (b) the presence of one local vibration mode, involving the upper part of the tower, that was clearly identified from the response data collected for more than 24 hours on the building [Gentile and Saisi, 2014].

It is further noticed that a wooden roof was installed in the upper part of the tower when the inner access was re-established through provisional scaffoldings. This roof, although very light, is slightly inclined in the N-E/S-W direction (as schematically shown in Fig. 4) and directly supported by the weakest structural part of the building. It is worth underlining that the roof slope, the redundant connection with the masonry walls and the thermal effects might induce not negligible thrusts on structural walls, that are very vulnerable due to the extensive decay, the presence of several discontinuities and the lack connection of the different adding.

3. **Preliminary ambient vibration tests**

Two 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 the metallic scaffolding and wooden roof, to check the possible effects of those additions on the dynamic characteristics of the structure.

Figure 4 shows the sensor layout adopted in the second test, with the accelerometers being installed on the inner side of the walls; it should be noticed that the same cross-sections were instrumented in the first AVT but the sensors were mounted on the outer side of the walls because the inner access to tower was not yet available in Summer 2012.

Since the Mantua historic centre is closed to road traffic and low level of ambient excitation had to be expected, the tower response was measured using high sensitivity accelerometers (WR model 731A, 10 V/g sensitivity). 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 of masonry towers. 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).

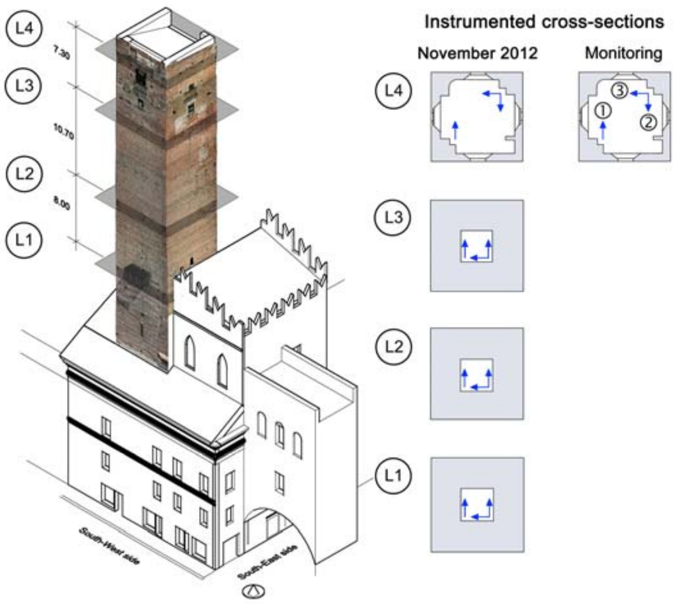


Fig. 4. *Instrumented cross-sections and sensors layout during the dynamic tests performed on November 2012 and the continuous dynamic monitoring*
Fig. 4. Sezioni strumentate e posizione dei sensori durante le indagini dinamiche condotte nel Novembre 2012 e durante il monitoraggio dinamico permanente

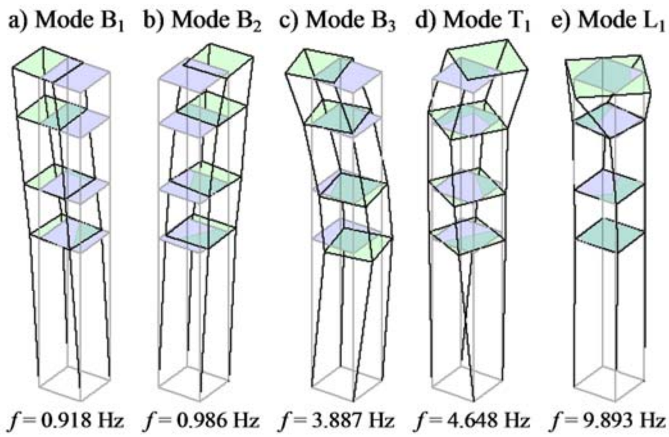


Fig. 5. *Vibration modes identified (SSI-Data) on 27/11/2012*
Fig. 5. Modi di vibrare identificati (SSI-Data) durante le indagini del 27/11/2012

The modal identification was performed by using a time window of 3600 s. The data-driven Stochastic Subspace Identification (SSI-Data) technique [van Overschee and de Moor, 1996] available in the commercial software ARTeMIS [SVS, 2012] was used to evaluate the modal parameters from response data.

The results of the second AVT, in terms of identified natural frequencies and mode shapes, are shown in Figure 5 and can be summarised as follows:

1. two closely spaced modes were identified in the range 0.90-1.00 Hz. These modes (Figs. 5a-b) are dominant bending (B) and involve bending in the two main planes of the tower, respectively;
2. the third mode (Fig. 5c) involves dominant bending in the N-E/S-W plane, with slight components also in the orthogonal plane;
3. the fourth mode (Fig. 5d), involves dominant torsion (T) of the tower until the height of 46 m and coupled torsion-bending of the upper level;
4. the last mode (Fig. 5e) is local (L) and characterized by torsion of the upper part of the tower.

The comparison with the dynamic characteristics identified in the first AVT [Gentile and Saisi, 2014] highlights that the mode shapes of lower modes (B_1-B_3) did not show significant changes, whereas the higher modes exhibited major changes at the upper level, that were conceivably related to the added wooden roof. The roof, even if very light, conceivably acts as a mass added in a vulnerable area and affects the dynamic characteristics of the upper region of the building. It is further noticed that differences in terms of natural frequency were also detected between the two tests, although (as it will be shown in the next section) conceivably related to temperature effects.

It is worth mentioning that, as expected from other studies [Ramos *et al.*, 2010; Cabboi *et al.*, 2013] reported in the literature, the mode shapes of Figure 5 highlights the possibility of identifying the tower vibration modes from the responses collected at the upper instrumented level. On the other hand, just measuring the structural response at the upper level provides information mainly on the natural frequencies, whereas more extensive instrumentation (see e.g. the sensor layout adopted on 27/11/2012, Fig. 4) is needed to obtain information also on the mode shapes along the height of the tower.

4. **Continuous dynamic monitoring system and data analysis**

Few weeks after the execution of the second AVT, a simple dynamic monitoring system was installed on the tower. The system is composed by:

(a) one 4-channels data acquisition system;
(b) 3 piezoelectric accelerometers (10 V/g sensitivity), mounted on the cross-section at the crowning level of the tower (Fig. 4);
(c) one temperature sensor, installed on the S-W front and measuring the outdoor temperature on the wall;
(d) one industrial PC on site, for the system management and data storage.

A binary file, containing 3 acceleration time series and the temperature data, is created every hour, stored in the local PC and transmitted to Politecnico di Milano for being processed.

The continuous dynamic monitoring system has been active since 17/12/2012. As in the preliminary tests, modal identification was performed using time windows of 3600 s, in order to comply with the widely agreed recommendation of using an appropriate duration of the acquired time window (ranging between 1000 e 2000 times the fundamental period of the structure, see e.g. [Cantieni, 2005]) to obtain accurate estimates of the modal parameters from OMA. In fact, OMA methods 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.

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.

The data files received from the monitoring system are managed in LabVIEW, where the following tasks are automatically performed [Busatta, 2012]: (a) creation of a database with the original data; (b) preliminary pre-processing (i.e. de-trending, automatic recognition and extraction of possible seismic events, creation of 1 dataset per hour); (c) evaluation of hourly-averaged acceleration amplitudes and temperature; (d) low-pass filtering and decimation of each dataset; (e) creation of a second database, with essential data records, to be used in the modal identification phase.

The modal identification was performed by applying the automated covariance-driven SSI procedure (SSI-Cov) [Peeters, 2000] developed in [Cabboi, 2013].

It is worth mentioning that the natural frequency estimates have been verified also by inspecting the first Singular Value (SV) line of the spectral matrix, which is the mode indication function adopted in the Frequency Domain Decomposition (FDD) method [Brincker *et al.* 2000]. Typical results of OMA in terms of natural frequencies can be summarized through the plot of Figure 6, showing the first SV line of the spectral matrix and the stabilization diagram obtained by applying the FDD and the automated SSI-Cov technique to the dataset recorded on 08/02/2013, 8:00-9:00. The inspection of Figure 6 clearly highlights that the alignments of the stable poles in the stabilization diagram of the SSI-Cov method provides a clear indication of the tower modes and those alignments of stable poles correspond to well-defined local maxima in the first SV line of the FDD technique.

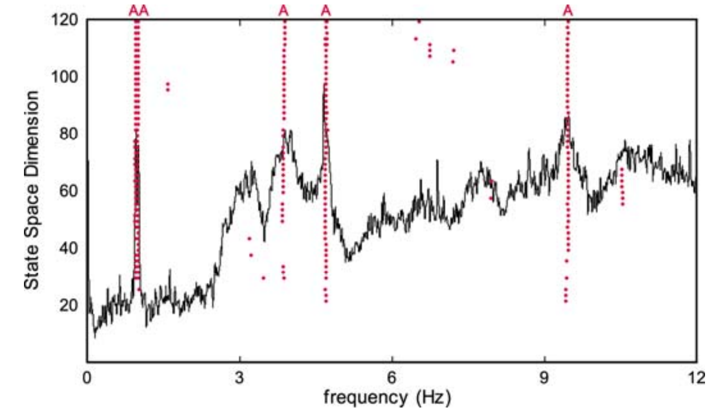


Fig. 6. *Typical stabilization diagram and automated (A) identification of the natural frequencies obtained by applying the SSI-Cov technique to the collected data (08/02/2013, 08:00-09:00)*
Fig. 6. Esempio di diagramma di stabilizzazione ed identificazione automatica (A) delle frequenze naturali ottenuti dall'applicazione della tecnica SSI-Cov ai dati registrati (08/02/2013, 08:00-09:00)

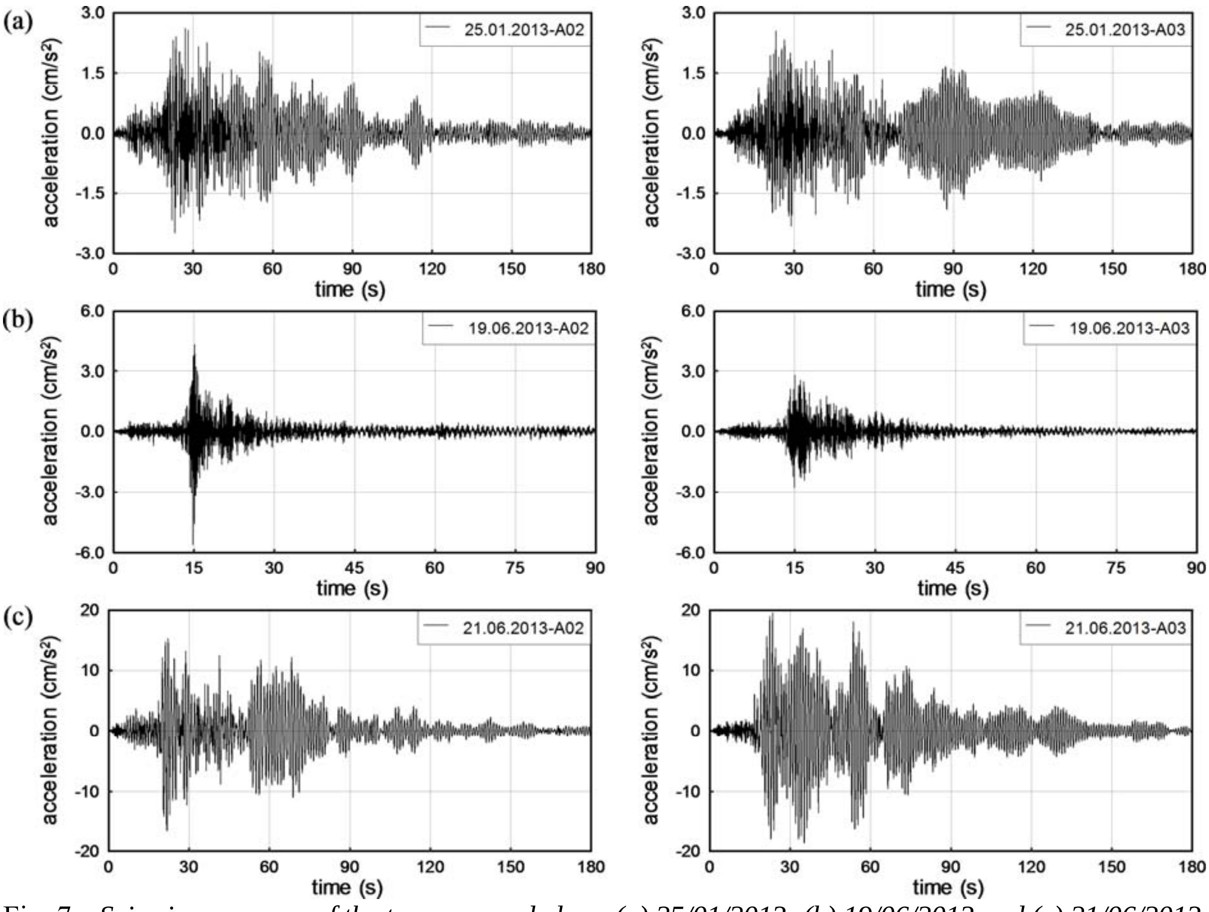


Fig. 7. *Seismic responses of the tower recorded on: (a) 25/01/2013; (b) 19/06/2013 and (c) 21/06/2013*
Fig. 7. Risposte ad eventi sismici registrate in data: (a) 25/01/2013; (b) 19/06/2013 e (c) 21/06/2013

Between January and June 2013, the monitoring system acquired the tower's response to different earthquakes occurred in the neighbouring regions. Figure 7 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. 7c, 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 next section.

5. **Continuous dynamic monitoring results**

5.1 *Dynamic characteristics under ambient excitation*

This section summarizes the main results of the dynamic monitoring obtained in a period of 8 months, from 17/12/2012 to 13/08/2013, using the automated SSI-Cov procedure described in [Cabboi, 2013] for modal identification; a total number of 5760 datasets were collected in the investigated time period.

Figure 8 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 Figure 9, whereas the corresponding statistics of natural frequencies are summarized in Table 1. This table includes the mean values (f_{ave}), the standard deviations (σ_f), and the extreme values ($f_{\min}$, $f_{\max}$) for all natural frequencies. It should be noticed that standard deviations are larger than 0.03 Hz for all global modes and especially significant for the local mode L_1, whose natural frequency varies from about 8.30 to 10.33 in 8 months.

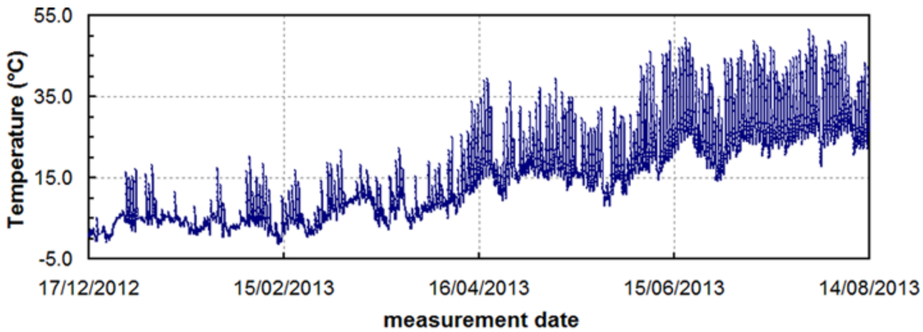


Fig. 8. *Outdoor temperature measured on S-W front in the period between 17/12/2012 and 13/08/2013*
Fig. 8. Temperatura esterna misurata sul fronte S-W nel periodo compreso tra il 17/12/2012 ed il 13/08/2013

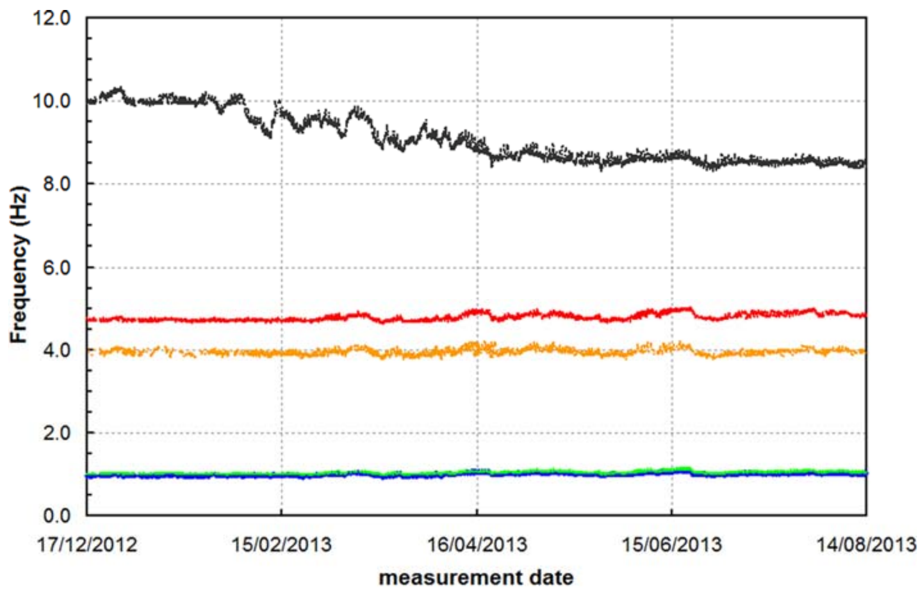


Fig. 9. *Variation of the automatically identified (SSI-Cov) natural frequencies in the period between 17/12/2012 and 13/08/2013*
Fig. 9. Variazione delle frequenze naturali identificate automaticamente (SSI-Cov) nel periodo compreso tra il 17/12/2012 ed il 13/08/2013

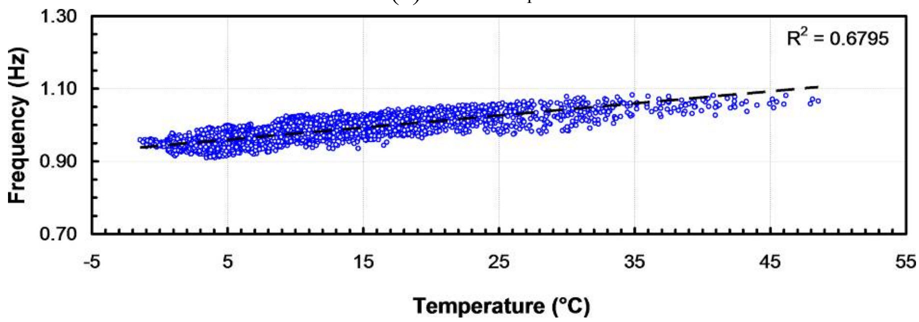
The inspection of Figures 8 and 9 firstly suggests that the slight fluctuation of the natural frequencies of global modes (B_1-B_3 and T_1, Figs. 5a-d) follows the temperature variation. In order to better demonstrate the effect of changing temperature on the fluctuations of the modal frequencies, the simplest approach is to plot each frequency with respect to temperature. For example, Figure 10 shows the natural frequencies of modes B_1 and B_2 plotted with respect to temperature along with best fit lines. The plots in Figure 10, referring to the period from 17/12/2013 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 [Gentile and Saisi, 2014]: the natural frequency of global modes tends to increase with increased temperature. This behaviour, observed also in other studies on masonry towers [Ramos *et al.*, 2010; Cabboi *et al.*, 2013; Cantieni, 2014] 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.

Table 1. *Statistics of the automatically identified (SSI-Cov) natural frequencies in the period between 17/12/2012 and 13/08/2013*

Tabella 2. Indici statistici di variabilità delle frequenze naturali identificate (SSI-Cov) automaticamente nel periodo compreso tra il 17/12/2012 ed il 13/08/2013

Mode	f_{ave} (Hz)	σ_f (Hz)	f_{min} (Hz)	f_{max} (Hz)
1 (B_1)	0.989	0.035	0.910	1.102
2 (B_2)	1.029	0.031	0.961	1.148
3 (B_3)	3.940	0.073	3.758	4.194
4 (T_1)	4.768	0.082	4.621	5.010
5 (L_1)	9.076	0.573	8.298	10.327

(a) Mode B_1



(b) Mode B_2

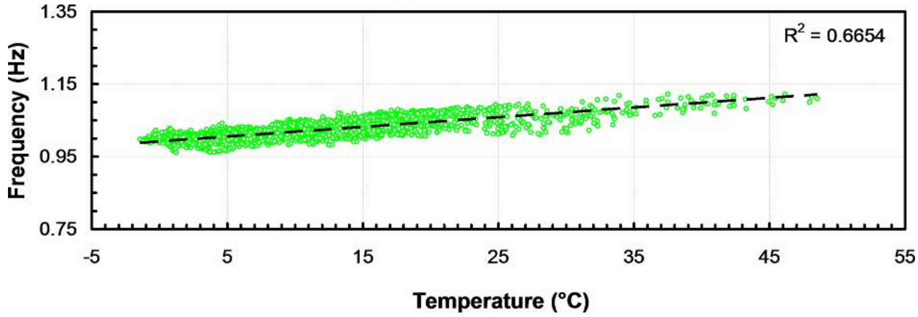


Fig. 10. *Natural frequency of modes B_1 (a) and B_2 (b) plotted with respect to the outdoor temperature on S-W front in the period between 17/12/2012 and 14/06/2013*

Fig. 10. Correlazione tra le frequenze proprie dei modi B_1 (a) e B_2 (b) e la temperatura esterna sul fronte S-W nel periodo compreso tra il 17/12/2012 ed il 14/06/2013

The time evolution of the natural frequency of the upper mode, i.e. the local mode L_1 (Fig. 5e), deserves some concern because the trend of this frequency is very different from the others (Fig. 9): 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.

A careful inspection of Figure 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 Figure 11. This figure 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. In other words, the average modal stiffness (i.e. the natural frequency) of the local mode decreases as both the average daily temperature and the daily temperature range tend to increase. This behaviour suggests the progress of a possible damage mechanism, conceivably related to the increase of the thrust exerted by the inclined roof on the supporting walls with increased temperature, and again confirms the poor structural condition and the high vulnerability of the upper part of the tower.

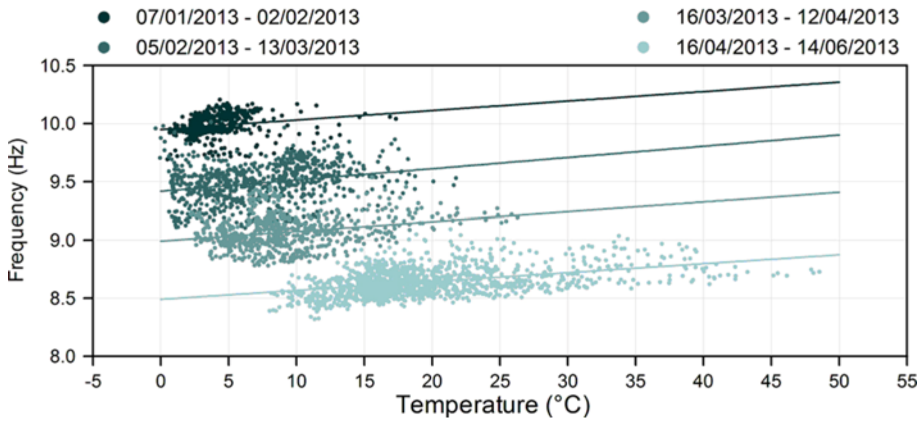


Fig. 11. *Natural frequency of local mode L_1 plotted with respect to the outdoor temperature in the period between 07/01/2013 and 14/06/2013*

Fig. 11. Correlazione tra la frequenza propria del modo locale L_1 e la temperatura esterna sul fronte S-W nel periodo compreso tra il 07/01/2012 ed il 14/06/2013

5.2 *Response to far-field seismic events*

As previously pointed out, data sets related to far-field seismic events were recorded on 25/01/2013 (Fig. 7a), 04/05/2013, 19/06/2013 (Fig. 7b) and 21/06/2013 (Fig. 7c). The latter event, corresponding to a significant earthquake occurred in the Garfagnana region, determined acceleration responses (Fig. 7d) on the top of the tower, that were more than 40 times larger than the usual ambient vibration responses.

In order to understand the tower behavior during the seismic events, a time-frequency analysis was performed, using the Matlab routine developed by [Ubertini *et al.*, 2013]. The routine, based on the Choi-Williams time-frequency distribution, allows to plot the signal's energy distribution in the time-frequency domain.

The variation in time of the frequency content (0-1.5 Hz) of the acceleration responses recorded during the earthquake of 21/06/2013 is shown in Figure 12. More specifically, Figures 12a and 12 b refer to the response of the tower at Ch. 2 (i.e. in the bending direction of mode B_1, Fig. 5a) and Ch. 3 (i.e. in the bending direction of mode B_2, Fig. 5b) of Figure 4, respectively. Both plots highlight a quite complex behavior, that is characterized by a clear decrease of the dominant frequencies, occurring just after the maximum accelerations have been reached and suggesting the occurrence of slight non-linearity.

Zooming the time evolution of the natural frequencies of lower modes B_1 and B_2, as shown in Figure 13, 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.

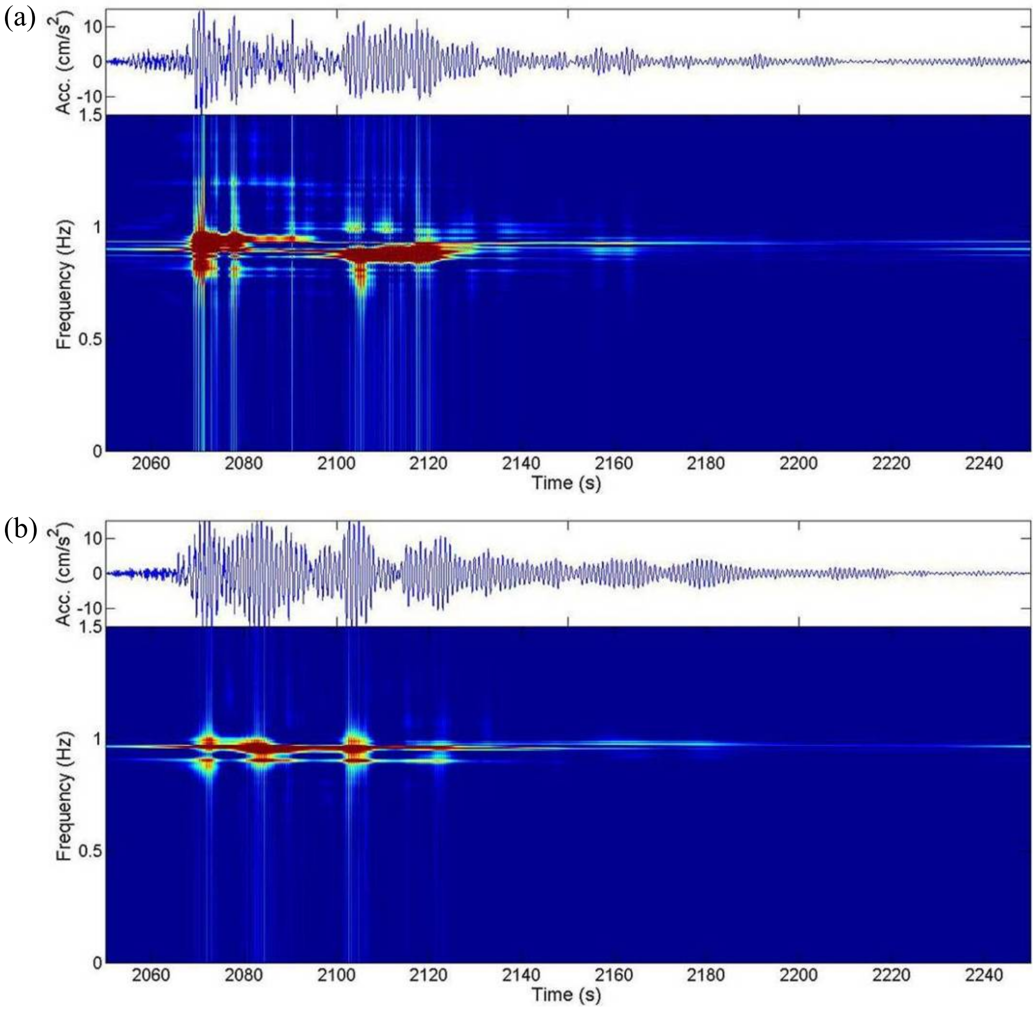


Fig. 12. *Time-frequency analysis of the time-histories recorded during the seismic event of 21/06/2013 at: (a) Ch. 2 (Fig. 4, N-E/S-W direction) and (b) Ch. 3 (Fig. 4, N-W/S-E direction)*

Fig. 12. Analisi tempo-frequenza delle risposte registrate durante l'evento sismico del 21/06/2013 in: (a) Ch. 2 (Fig. 4, direzione N-E/S-W) e (b) Ch. 3 (Fig. 4, direzione N-W/S-E)

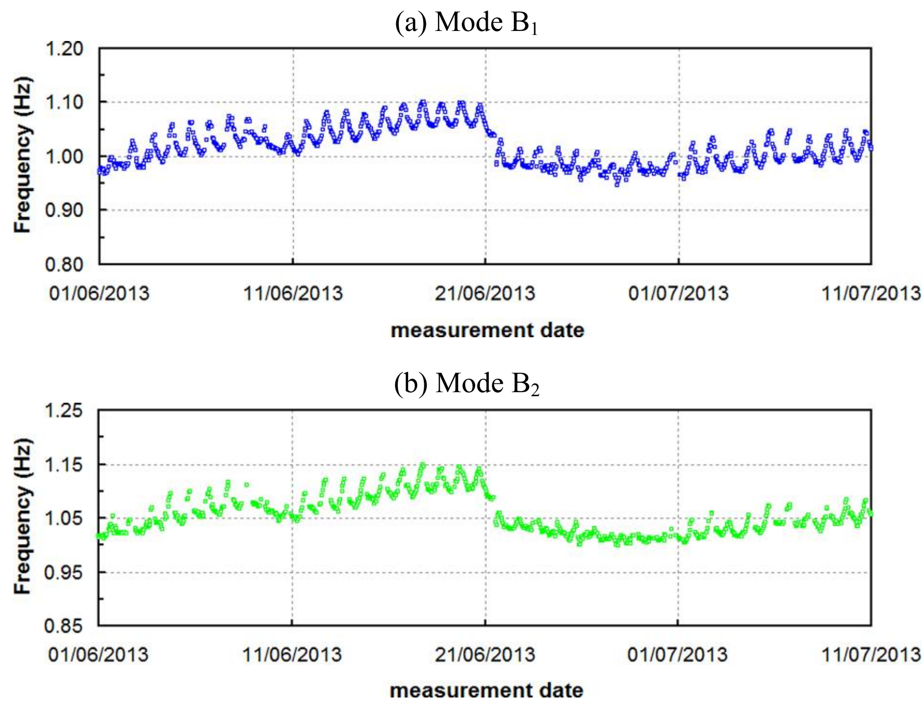
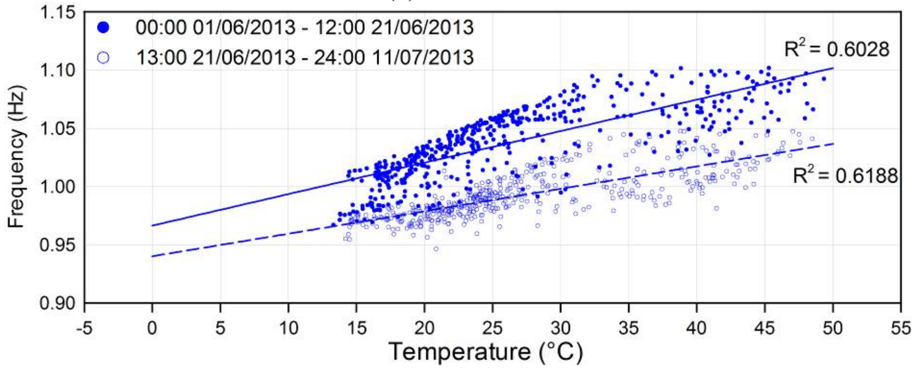


Fig. 13. *Variation of the natural frequency of modes B_1 (a) and B_2 (b) in the period between 01/06/2013 and 10/07/2013*

Fig. 13. Variazione delle frequenze proprie dei modi B_1 (a) e B_2 (b) nel periodo compreso tra il 01/06/2013 ed il 10/07/2013

The results of this check are summarized in Figure 14, 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 frequency shifts are non reversible.

(a) Mode B_1



(b) Mode B_2

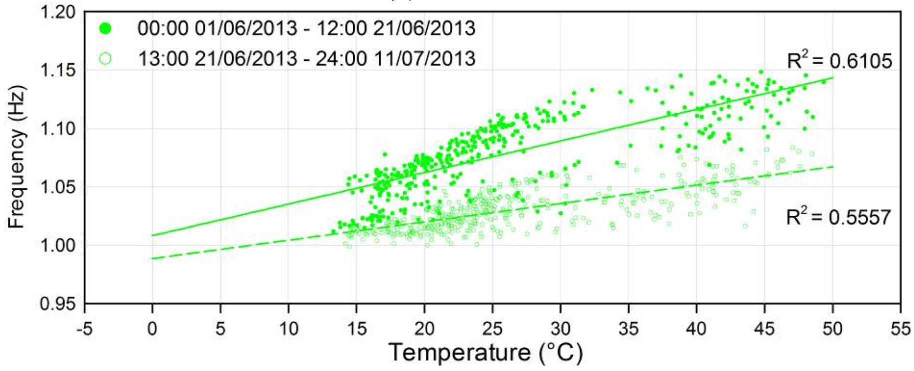


Fig. 14. *Change in the frequency-temperature correlation induced by the seismic event of 21/06/2013: (a) Mode B_1; (b) Mode B_2*

Fig. 14. Variazione della correlazione frequenza-temperatura a seguito dell'evento sismico del 21/06/2013: (a) Modo B_1; (b) Modo B_2

6. **Conclusions**

The paper focuses on the post-earthquake assessment of a historic masonry tower in Mantua, Italy and summarizes the results of visual inspection, preliminary ambient vibration tests and continuous dynamic/seismic monitoring of the building.

Visual inspection of 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 in the first dynamic test [Gentile and Saisi, 2014].

Those results highlighted the need for preservation actions to be carried out. Hence, a light wooden roof and a metallic scaffolding were installed in the tower to allow both the inspection of inner load-bearing walls in view of the planned strengthening and the installation of a continuous dynamic monitoring system. In order to check the effect of both wooden roof and scaffolding on the dynamic characteristics 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:

- the mode shapes of the lower vibration modes are practically unchanged;

- the first torsion mode changed its mode shape in the upper part of the building. More specifically, the torsion component continues to be dominant in the lower part of the tower, whereas the upper region is characterized by coupled bending and torsion;
- 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 inside the building, as a part of the health monitoring process aimed at helping the preservation of the historic structure.

The time evolution of the modal frequencies automatically identified between 17/12/2012 and 13/08/2013 (8 months, 5760 datasets of 1 hour each) clearly highlights:

1. the effect of temperature on natural frequencies of the vibration modes. More specifically, the frequencies of the global modes increase with increased temperature probably as consequence of the closure of superficial cracks, minor masonry discontinuities or mortar gaps induced by the thermal expansion of materials;
2. the quick progress of a possible damage mechanism, involving the upper part of the building, and clearly identified through the remarkable fluctuations and the significant decrease (about 15% in 8 months) of the natural frequency corresponding to the local mode. In the authors' opinion, this anomalous behaviour is conceivably related to the thrust determined by the thermal expansion of the wooden roof on the dismantled masonry characterizing the top region of the tower;
3. the non-reversible decrease of the natural frequencies of the fundamental modes detected after the occurrence of a far-field seismic event (21/06/2013). The evidence of this decrease is provided by both the time-frequency analysis of the tower's response during the earthquake and the comparison of the modal peaks identified before and after the seismic event. Furthermore, the inspection of the frequency-temperature relationships (including data collected in the 3 weeks before and after the earthquake) confirms that the frequency-temperature regression lines exhibited a remarkable variation after the seismic event.

As a final remark, it is worth underlining the practical feasibility of damage detection methods based on natural frequencies shifts (provided that the temperature effects are accounted for) and the key role of dynamic monitoring in the diagnosis of the investigated tower. Of course, a more extensive instrumentation along the structure's height would have provided much more information on the possible damage mechanism involving the upper region and the position of the earthquake-induced damage.

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