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OMA-BASED STRUCTURAL HEALTH MONITORING OF HISTORIC STRUCTURES

Carmelo Gentile[1], and Antonella Saisi[2]

[1] Professor, Department ABC, Politecnico di Milano, Milan, Italy, carmelo.gentile@polimi.it
[2] Assistant Professor, Department ABC, Politecnico di Milano, Milan, Italy, antonella.saisi@polimi.it

ABSTRACT

Structural Health Monitoring (SHM) is generally defined as a multi-disciplinary process involving: (a) the repeated or continuous measurement of the response of a structural system through arrays of appropriate sensors; (b) the extraction from measured data of features, which are representative of the health condition and (c) the statistical analysis of these features to detect any novelty or abnormal change in the investigated system. Among the different SHM strategies, the fully non-destructive nature and the minimum impact of the vibration monitoring makes the OMA-based approach especially suitable to address the preservation of Cultural Heritage structures and also to avoid, in some cases, inappropriate strengthening interventions. In the paper, the main ideas of OMA-based SHM of historic structures are presented and exemplified through the application to both relatively simple and very complex buildings, such as towers and cathedrals.

Keywords: Automated OMA, Environmental effects, Historic structures, Masonry towers, SHM

1. INTRODUCTION

In recent years, the Structural Health Monitoring (SHM) methodology based on continuous dynamic monitoring of structures and output-only modal identification has received increasing attention and the installation of dynamic monitoring systems, especially on bridges [1-3], has become more common. Among the many motivations for the raising scientific interest on dynamic monitoring, the most relevant are: (a) the ageing of existing structures (often accompanied by poor maintenance and harsh environmental conditions) as well as the increasing complexity of new constructions; (b) the technological advances, allowing more economical installation of monitoring systems and fully computer-based operation, with efficient transmission and processing of the collected data; (c) the possibility of performing the health condition assessment of a structure from the analysis of its dynamic response to ambient and/or operational excitation, with no need for artificial inputs.

Although the minimum impact of the vibration monitoring makes this technology especially suitable to the context of preservation of Cultural Heritage structures, the practical applications in this field are still quite limited [4-19]. A large part of published studies [4-9], [11], [14-17] focuses on the ancient towers, that represent a significant part of the existing Cultural Heritage buildings, as these historic structures were built over the centuries with different characteristics and functions: bell towers, lookout or defensive towers, chimneys and minarets.

The idea of performing cost-effective OMA-based SHM of ancient towers has been taking shape recently [4-9], [11], [14-17] as those structures are generally sensitive to ambient excitation and exhibit a cantilever-like behavior, so that the successful monitoring of the dynamic characteristics can be obtained by permanently installing a few high-sensitivity accelerometers (or seismometers) in the upper part of the building. In addition, masonry towers often exhibit high vulnerability to seismic actions, so that the possibility of using a limited number of sensors to provide an effective condition monitoring is of utmost importance to promote extensive and sustainable assessment programs. On the other hand, the availability of a limited number of sensors imposes the choice of resonant frequencies as features to be assumed as representative of the structural condition. Since resonant frequencies are also sensitive to factors other than structural changes, a vibration-based strategy for the preventive conservation of masonry towers should include:

1. Dynamic tests in operational conditions, carried out using an appropriate number of sensors in order to identify the dynamic characteristics of the tower and to highlight the most meaningful positions (among the possible ones) to be permanently instrumented;
2. Continuous monitoring and SHM, based on the installation of a limited number of sensors in the structure (at least 3 accelerometers and 1 temperature sensor). Data should be collected periodically [4] or continuously [6-8], [11], [14-17] and (automated) operational modal analysis (OMA) should be carried out to track the evolution in time of resonant frequencies, understand the environmental effects and detect the occurrence of any structural performance anomaly [7-8, 14-17];
3. After the SHM has revealed a significant deterioration of the structural condition, local assessment of damage should be carried out through detailed visual inspection and complementary non-destructive tests in order to plan strengthening interventions with more confidence, only if and where the interventions are really necessary.

The paper firstly exemplifies the application of the previous tasks (1) and (2) to three historic masonry towers, with the main objectives of understanding the possible "normal" changes of natural frequencies associated to environmental effects. The three investigated towers are the *Gabbia tower* in Mantua [7-8], the *San Vittore bell-tower* in Arcisate, Varese [14] and the *Santa Maria del Carrobiolo bell-tower* in Monza [17]. In all cases, the installation of the monitoring devices was preceded by historic and documentary research, visual inspection, topographic survey, non-destructive and minor-destructive tests of materials on site.

In the last part of the paper, the focus is moved to the vibration monitoring of complex Cultural Heritage buildings, requiring the installation of a large number of sensors and SHM strategies conceivably involving all the available modal parameters [20]. The main expectations are exemplified by using data collected in the continuous monitoring of the Milan Cathedral [18].

2. SHM OF ANCIENT TOWERS USING FEW ACCELEROMETERS

As previously pointed out, the application of OMA-based SHM is exemplified to three historic masonry towers [7-8], [14], [17]; in all the presented case studies, continuous dynamic monitoring systems, including few accelerometers and temperature sensors, were installed in the upper part of the buildings and automatic modal identification was performed. The identified natural frequencies clearly show a time evolution over the monitoring period due to the changing environment.

The dynamic monitoring systems were composed by: (a) one 4-channels data acquisition system (24-bit resolution and 102 dB dynamic range); (b) 3-4 high-sensitivity accelerometers; (c) at least one temperature sensor and (d) one industrial PC on site, for system management and data storage. Binary files, containing the acceleration time series and the temperature data, are created every hour, stored in the local PC and transmitted to Politecnico di Milano for being processed. The sampling frequency was 200 Hz, which is much higher than that required for the investigated structures; consequently, low pass filtering and decimation were applied to the data before the use of the modal identification tools.

The data files received from the monitoring systems are managed in LabVIEW, where the following tasks are automatically performed [21]: (a) creation of a database with the original data; (b) preliminary pre-processing (i.e. de-trending, automatic recognition and extraction of the time series corresponding to swinging of bells or possible seismic events); (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 for automated OMA. Finally, the modal parameters of the towers are extracted from the measured acceleration data using an automated procedure [22], based on the covariance-driven Stochastic Subspace Identification (SSI-Cov) algorithm (see e.g. [23]).

2.1. Detecting damage under changing environment: the *Gabbia* Tower

The *Gabbia* tower [7-8] (Fig. 1), with its 54.0 m height, is the tallest tower in Mantua. The tower dates back to the 13th century and the structure is built in solid brick masonry, with the load-bearing walls being about 2.4 m thick, except in the upper levels, where the walls thickness decreases to about 0.7 m. As shown in Fig. 1, the tower is nowadays part of an important palace, whose load-bearing walls seem to be not effectively connected to the tower; nevertheless, several vaults and floors of the palace are directly supported by the tower. While the main part of the building – below the height of about 46.0 m from the ground level – did not exhibit any evident structural damage (with the materials being only affected by superficial decay), the upper part of the tower is in a poor state of preservation. In more details, at a distance of about 8.0 m from the top, the brick surface workmanship changes and the masonry quality significantly decreases (as it was confirmed by pulse sonic tests); furthermore, the presence of several structural discontinuities and the lack of mechanical connection between subsequent addings determined concerns on the seismic behavior of the upper part of the tower.

After preliminary ambient vibration tests (November 2012, Fig. 2), a simple dynamic monitoring system was installed in the tower on December 17th, 2012 (and removed at the beginning of July 2015). The sensing devices consisted of: (i) three piezoelectric accelerometers (WR model 731A, 10 V/g sensitivity and ±0.50 g peak), mounted on the cross-section at the crowning level of the tower (Fig. 2a), and (ii) one temperature sensor, installed on the S-W front and measuring the outdoor wall temperature.

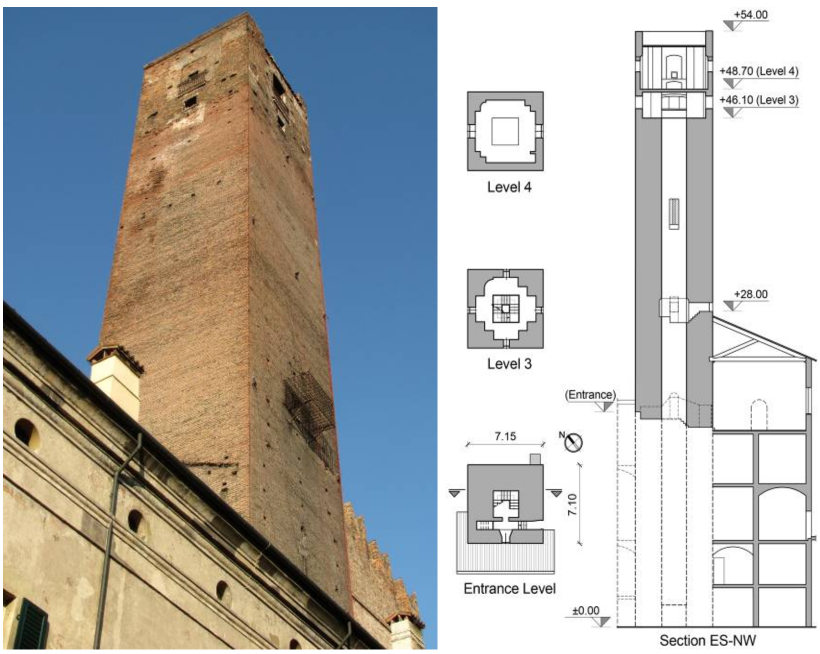

Figure 1. View of the *Gabbia* tower in Mantua, Italy and sections of the tower (dimensions in m).

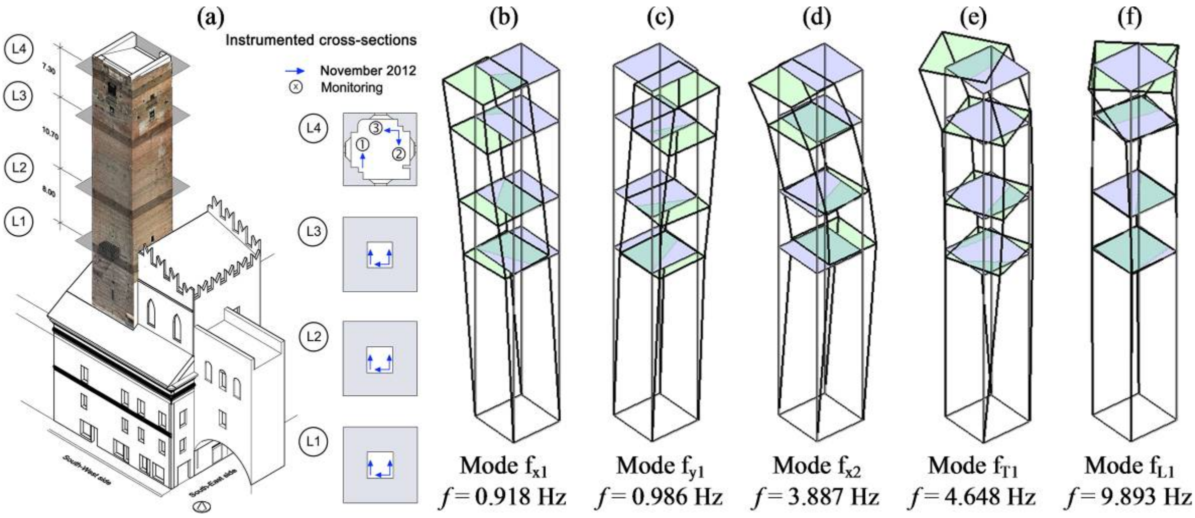

Figure 2. (a) Instrumented cross-sections and layout of the accelerometers during the preliminary tests (November 2012) and the continuous dynamic monitoring; (b)-(f) Identified modes of vibration.

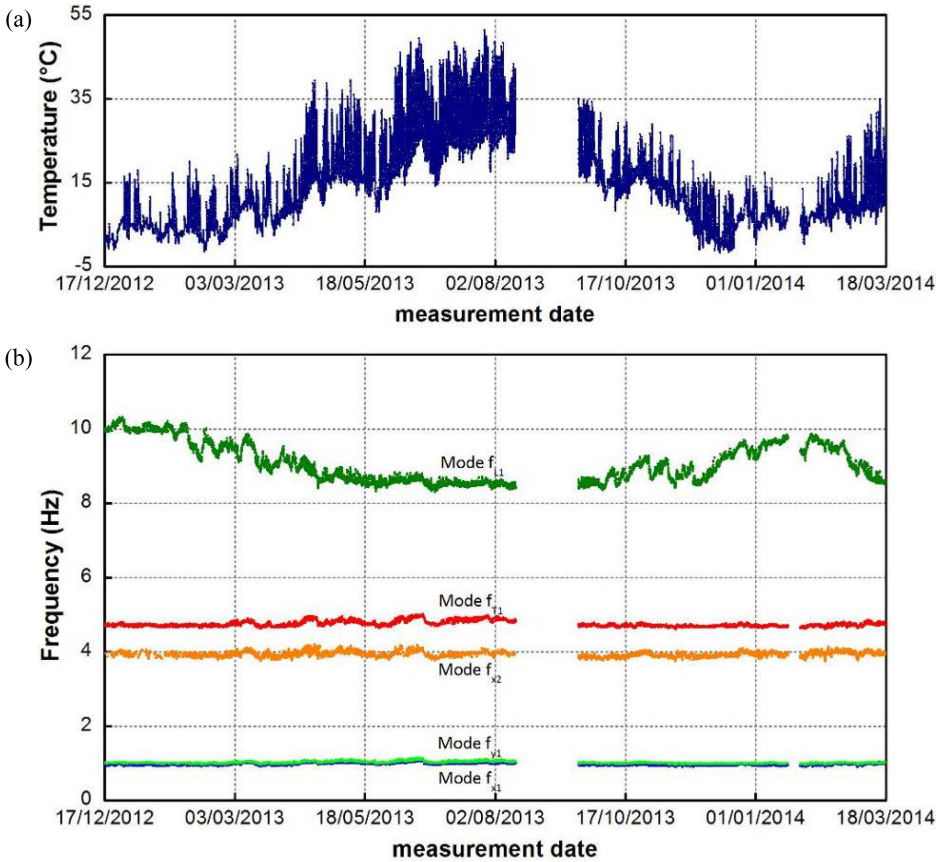

Figure 3. Time evolution (from 17/12/2012 to 17/03/2014) of: (a) the temperature measured on the S-W front of the tower; (b) the automatically identified natural frequencies.

Five vibration modes were identified in the preliminary ambient vibration tests and the corresponding natural frequencies were successfully tracked during the monitoring period. As shown in Fig. 2, the identified modes correspond to three bending modes (Figs. 2b-d), one torsion mode (Fig. 2e) and one local mode involving the upper part of the structure (Fig. 2f).

It should be noticed that, until June 2013, the tower's response to different far-field earthquakes was recorded. The strongest event – corresponding to an earthquake occurred in the Garfagnana region (Tuscany) on June 21st, 2013 – was characterized by a measured peak acceleration of about 20 cm/s^2, exceeding about 50 times the highest amplitude of normally observed ambient vibrations.

Figure 3 reports the time evolution of the outdoor temperature (S-W front, Fig. 3a) and of the automatically identified modal frequencies (Fig. 3b) for a period of about 15 months, from 17/12/2012 to 17/03/2014.

The temperature tracking reveals large fluctuations, between −2°C and +45°C, with significant daily variations on sunny days. A closer inspection of Fig. 3b highlights that the natural frequencies of global modes (f_{x1}, f_{y1}, f_{x2} and f_{T1}, Fig. 2) vary accordingly with the outdoor temperature. This correlation can be better investigated by plotting each modal frequency with respect to the recorded temperature. Figs. 4a-c show the results obtained for modes f_{x1}, f_{y1} and f_{T1} along with the best fit lines and the coefficient of determination R^2: the three plots refer to the time period from 17/12/2012 to 20/06/2013 and confirm that the frequency of global modes tends to increase with increased temperature. This behavior, observed in all studies of masonry towers can be explained by the closure of superficial cracks, minor masonry discontinuities or mortar gaps induced by the thermal expansion of materials [7].

The frequency-temperature relationships obtained after the Garfagnana earthquake (21/06/2013) are shown in Figs. 4d-f. The comparison with the results referred to the first six months of monitoring (Figs. 4a-c) reveals significant differences: the frequency dependence on temperature still remains roughly linear but the regression lines of all modes exhibit remarkable variations after the earthquake, with the temperature range being almost unchanged. This trend is also confirmed by the general decrease of the statistics of the natural frequencies (mean value, standard deviation, extreme values) summarized in Table 1 and highlights the occurrence of abnormal structural changes.

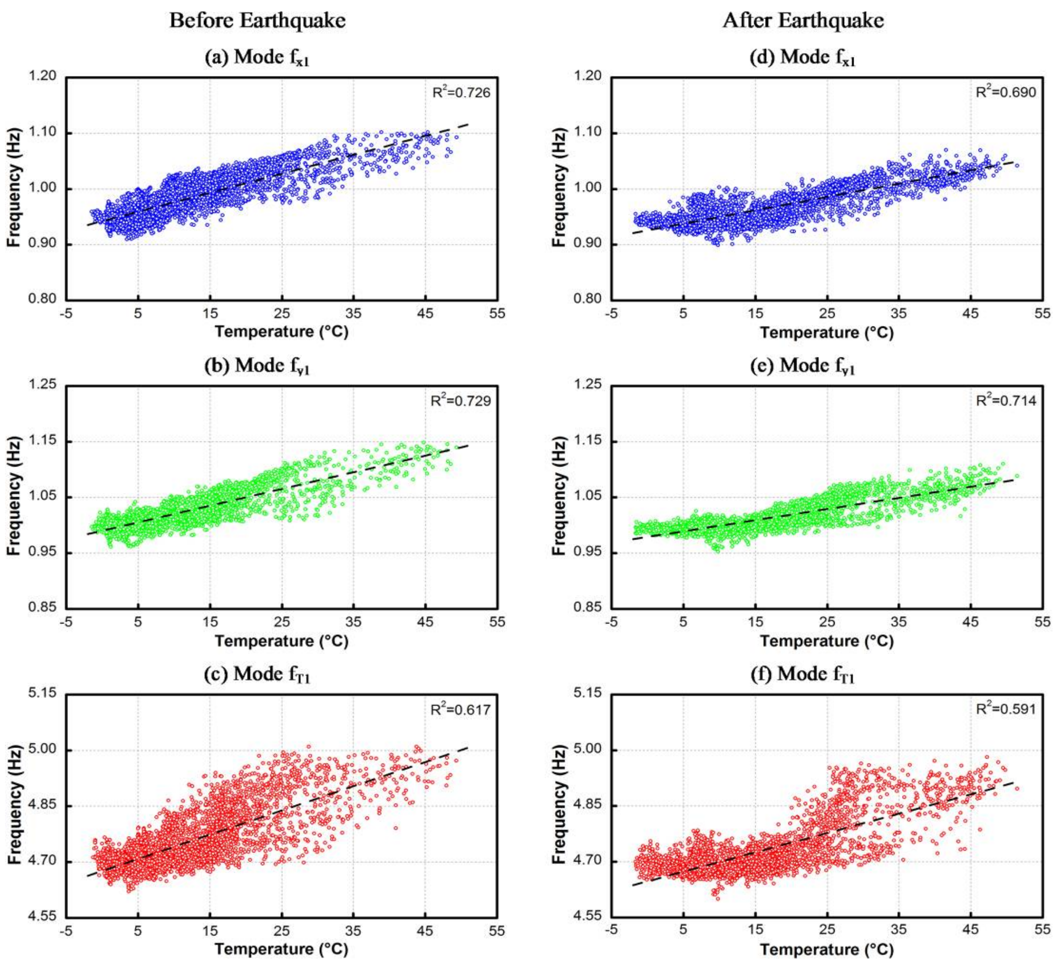

Figure 4. Natural frequency of modes f_{x1}, f_{y1} and f_{T1} plotted versus the outdoor temperature: (a)-(c) from 17/12/2012 to 20/06/2013; (d)-(f) after 21/06/2013.

Table 1. *Gabbia* tower : Statistics of the natural frequencies identified before and after the seismic event of 21/06/2013.

Mode	f_{ave} (Hz)		σ_f (Hz)		f_{min} (Hz)		f_{max} (Hz)	
	Before	After	Before	After	Before	After	Before	After
f_{x1}	0.985	0.968	0.038	0.031	0.910	0.897	1.102	1.070
f_{y1}	1.024	1.012	0.032	0.025	0.961	0.953	1.148	1.110
f_{x2}	3.941	3.929	0.075	0.063	3.758	3.742	4.194	4.137
f_{T1}	4.754	4.727	0.077	0.066	4.621	4.600	5.010	4.982
f_{L1}	9.222	8.937	0.554	0.433	8.385	8.332	10.327	9.862

It worth mentioning that very similar results have been more recently obtained during the seismic monitoring of the *San Pietro* bell-tower in Perugia [19] during the seismic sequence of Central Italy occurred in 2016. In both studies, clear permanent drops of natural frequencies have been observed after the seismic events and the frequency shifts were remarked by changes of the frequency-temperature relationships. In addition, even if the experimental data clearly reveal the occurrence of slight permanent changes [7-8], [19], the damage on the structures was not detectable through visual inspections.

In order to complete the discussion of the results of the *Gabbia* tower's monitoring with the local mode f_{L1}, Fig. 3b shows that frequency evolution of f_{L1} looks very different from the others and is characterized by significant frequency variations (approximately in the range 10.33-8.33 Hz); similarly, Table 1 confirms that high values of the standard deviation are especially observed for this mode. The observed behavior suggests the progress of a possible damage mechanism, conceivably related to the thrust exerted by the inclined wooden roof with increased temperature, and confirms the poor structural condition and the high vulnerability of the upper part of the tower. This conclusion seems to be confirmed by the frequency loss of the local mode detected after one year of monitoring (and after the earthquake), with the natural frequency being unable to reach the maximum values identified one year before in a similar temperature conditions (Fig. 3b).

2.2. Dynamic monitoring of the *San Vittore* bell-tower

The *San Vittore* bell tower [14] is located in the small town of Arcisate (northern Italy) and is connected to the neighboring church *Chiesa Collegiata di San Vittore* (11th century) on the east side and partly on the south side (Fig. 5). The tower, built in stonework masonry, is about 37.0 m high and has a square plan (5.8 m × 5.8 m); the thickness of the load-bearing walls progressively decreases from 135 cm at the ground level to 65 cm at the top.

The *San Vittore* bell tower has been studied by Politecnico di Milano since 2007: early studies included direct survey and testing of materials as well as ambient vibration tests and FE modeling [14]. Subsequently, a static monitoring system was installed in the tower; this system, aimed at collecting information on the evolution of crack patterns, included 15 extensometers and 8 temperature sensors, that measured internal and external temperature at different levels of the tower.

Two ambient vibration tests were conducted on June 2007 and June 2008. In both tests, the response of the tower was measured in 15 selected points (Fig. 6a) and datasets of 3600 s were collected adopting a sampling frequency of 200 Hz. Five vibration modes were identified in the frequency range 0-6 Hz (Fig. 6b-f). As expected, the identified modes can be classified as bending and torsion: dominant bending modes were identified at 1.22 (f_1, Fig. 6b), 1.28 (f_2, Fig. 6c), 4.01 (f_4, Fig. 6e) and 4.16 Hz (f_5, Fig. 6f) while only one torsion mode (f_3, Fig. 6d) was identified at 3.60 Hz. It is worth noting that the dominant bending modes of the tower (Figs. 6b-c and 6e-f) involve flexure practically along the diagonals.

In June 2009, a continuous dynamic monitoring system was installed in the bell tower, with the sensing devices consisting of 3 uni-axial Dytran 3191A1 piezoelectric accelerometers (10 V/g sensitivity and ±0.50 g peak). Logistical motivations and the results of the preliminary tests allowed to identify the level at 22.78 m (Fig. 6a) as the one to be instrumented in order to detect all the previously observed modes.

The continuous dynamic monitoring was carried out for 8 months (from June 2009 to February 2010), during which the monitoring system had to be stopped several times because of both normal maintenance and the installation of a new electrical system in the tower. It is worth mentioning that during the investigated time span, the static monitoring system installed in the tower not only provided temperature data to complement the dynamic monitoring but also indicated that no abnormal changes of the structural condition occurred.

Automated modal identification was performed using time windows of 2500 s [5], [14] which were obtained from each 1-hour recorded dataset after detection and removal of the time series corresponding to the ringing/swinging of bells. Hourly averaged temperature data were also recorded by a previously installed static monitoring system. Among the 8 available temperature measurements, correlation studies [5] indicated that the temperature recorded indoor on the East-side (T_{E1int}) and outdoor on the North-side (T_{N3ext}) are the most representative of the internal and external thermal condition of the tower, respectively. Hence, the temperature data T_{E1int} and T_{N3ext} were selected to investigate the correlation with the automatically identified natural frequencies.

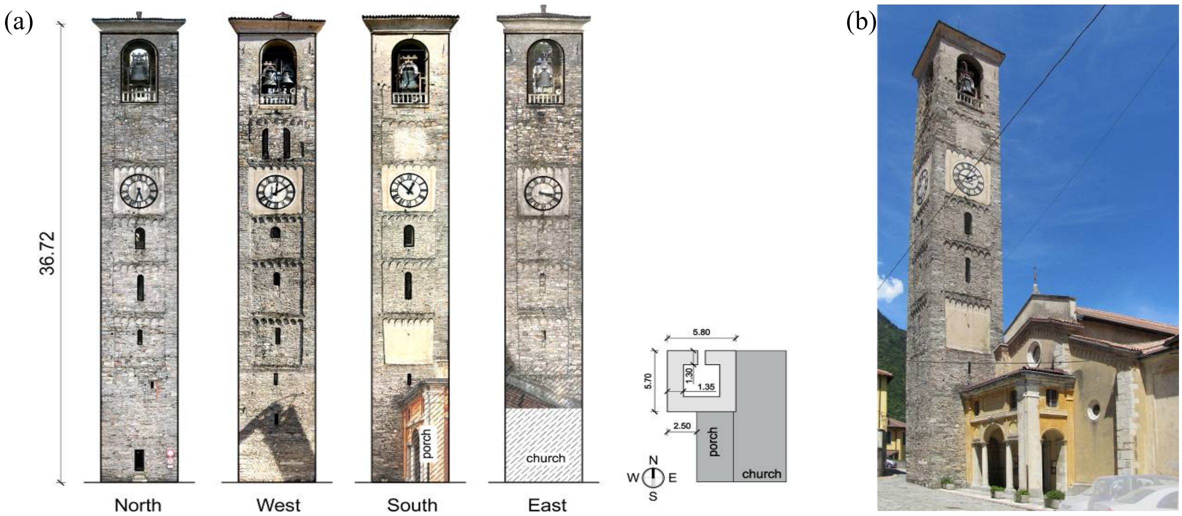

Figure 5. (a) Fronts and cross-section (dimensions in m) of the *San Vittore bell-tower*; (b) View of the tower.

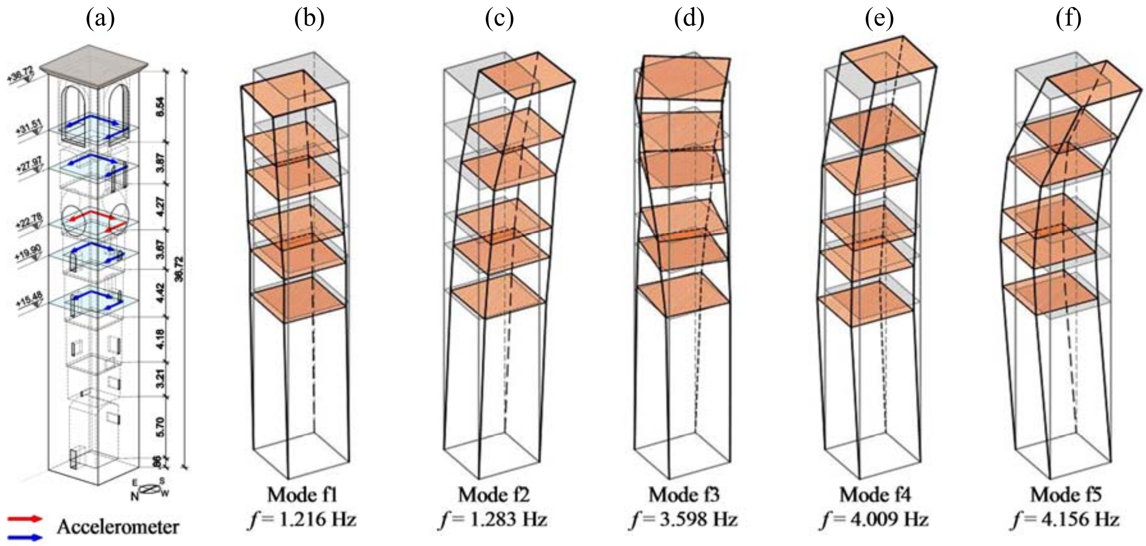

Figure 6. (a) Instrumented cross-sections and layout of accelerometers during the dynamic tests (blue and red arrows) and the continuous monitoring (red arrows); (b) Vibration modes identified from ambient responses.

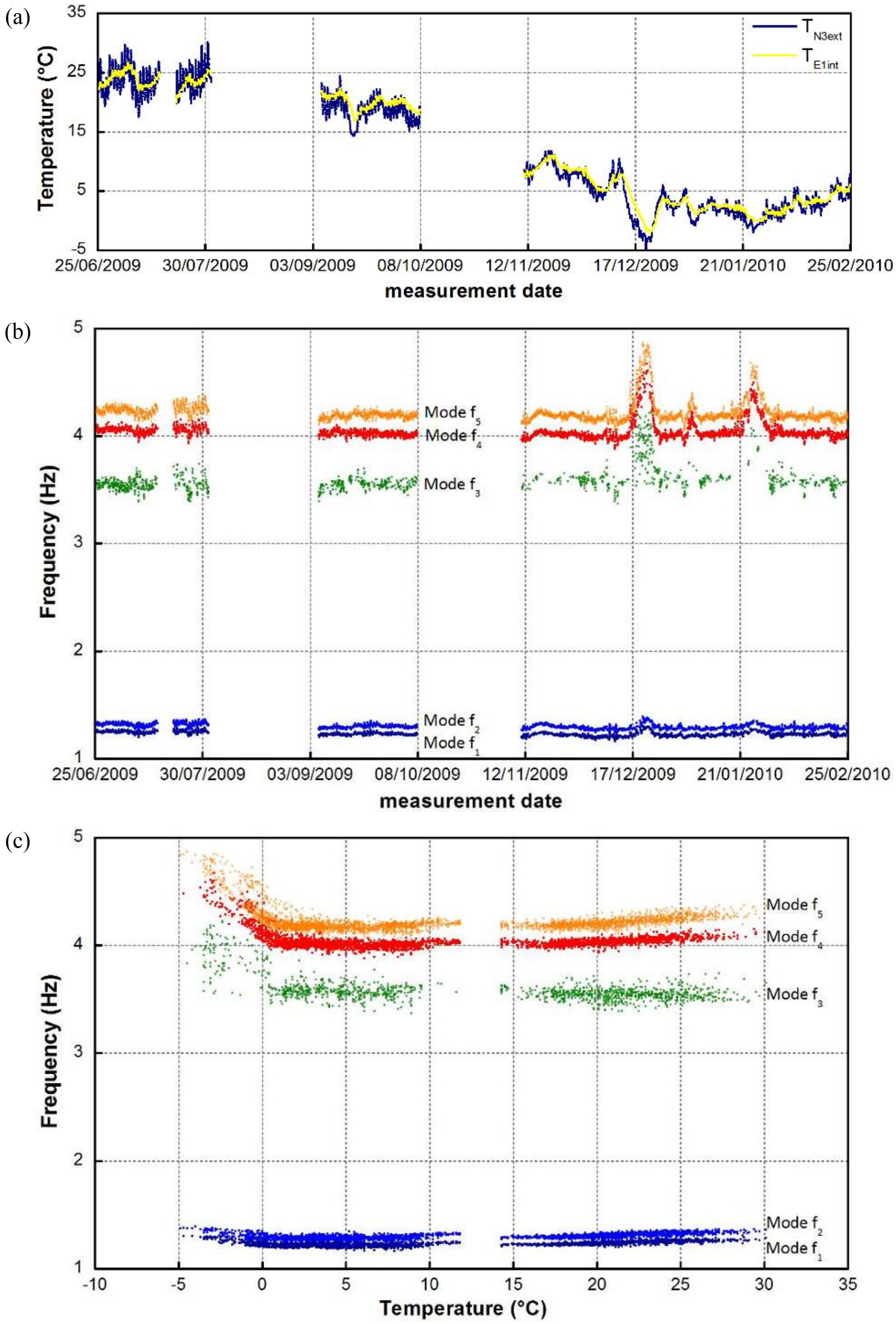

Figure 7. Time evolution (from 25/06/2009 to 24/02/2010) of the temperatures T_{N3ext} and T_{E1int}; (b) Time evolution (from 25/06/2009 to 24/02/2010) of the automatically identified natural frequencies; (c) Natural frequencies plotted with respect to the outdoor temperature (from 25/06/2009 and 24/02/2010).

Figure 7 shows the time evolution of temperatures (Fig. 7a) and natural frequencies (Fig. 7b) as well as the correlation between modal frequencies and the outdoor temperature T_{N3ext} (Fig. 7c). The statistics (mean value, standard deviation, minimum and maximum value) of the natural frequencies identified during the monitoring are reported in Table 2.

The inspection of Figs. 7a-c allows the following comments: (a) the bending modes are more frequently identified than the torsion mode, probably as a consequence of the low level of ambient excitation characterizing the tower; (b) similarly to the global modes of the *Gabbia* tower, the modal frequencies follow almost linearly the daily temperature variation, as their value increases with the increased temperature; (c) corresponding to below zero temperatures, the natural frequencies rapidly and significantly increase with decreased temperature. This last behavior, also observed in [11], is conceivably related to the freezing of the structural system (including its foundation) and to the presence of ice, which fills and closes the cracks, causing a temporary stiffening of the structure.

It is worth mentioning that, although the temperature effects on natural frequencies turned out to be quite complex for the *San Vittore* bell-tower, the well-known principal component analysis (PCA, see e.g. [16]) algorithm does provide a robust tool to remove the environmental effects and to develop effective prediction models of the identified frequencies for SHM purposes [14].

Table 2. *San Vittore* bell-tower: Statistics of the natural frequencies identified from 25/06/2009 and 24/02/2010.

Mode	f_{ave} (Hz)	σ_f (Hz)	f_{min} (Hz)	f_{max} (Hz)
f_1	1.228	0.019	1.162	1.290
f_2	1.297	0.021	1.240	1.367
f_3	3.552	0.056	3.371	3.774
f_4	4.023	0.033	3.880	4.184
f_5	4.196	0.042	4.043	4.372

2.3. Dynamic monitoring of the *Santa Maria del Carrobiolo* bell-tower

The last investigated tower belongs to the historic religious complex of *Santa Maria del Carrobiolo* [17] in Monza (Italy). The religious complex includes a church, a bell-tower, a monastery, an oratory and other minor buildings, which were erected at different times. The tower (Fig. 8a), about 33.7 m high, is built in solid brick masonry and has nearly square plan (5.93 m × 5.70 m); the thickness of the load bearing walls slightly decreases from 70 cm at the ground level to 58 cm at the top.

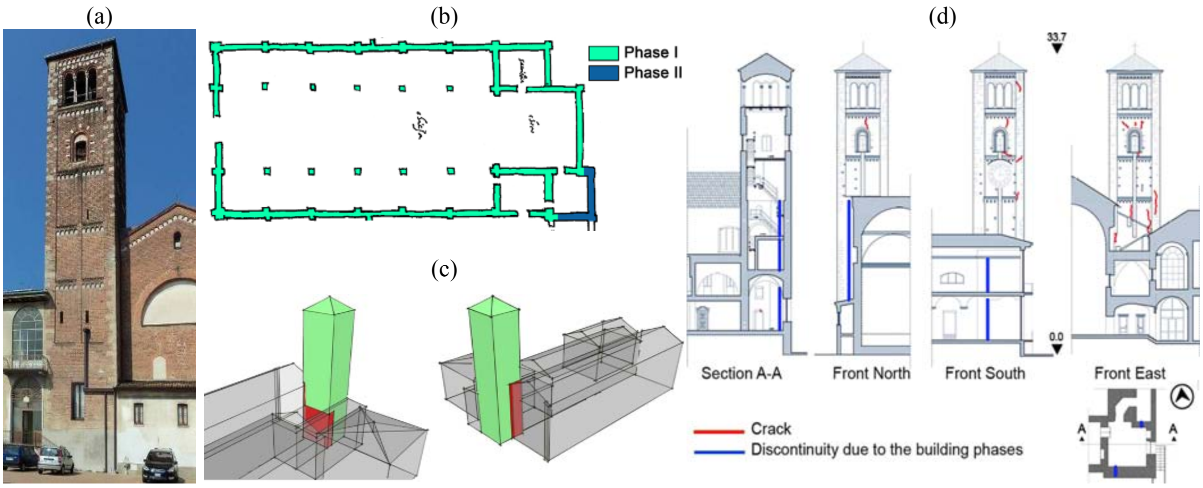

Figure 8. (a) View of the Santa Maria del Carrobiolo bell-tower (Monza, Italy); (b) Building phases in a plan dating back to 1572; (c) Schematic representation of the interaction between the bell-tower and the church apse; (d) Section and fronts of the bell-tower.

Historical documents testify that the construction of the church and monastery dates to the 13th century, whereas the bell-tower was completed in 1339 (Fig. 8b). The sequence of the construction stages has been confirmed by visual inspection of the masonry discontinuities: (a) the North and West sides of the tower are directly supported by the load-bearing walls of the apse and the right aisle of the church; (b) the Southern and Eastern load-bearing walls of the tower are continuous from the ground to the roof but do not exhibit any mechanical connections with the walls of the church (Figs. 8c and 8d). Moreover, several cracks cut the entire wall thickness, mainly at the level below the belfry (Fig. 8d), and a metallic tie-rod opposes to the opening of a deep crack on the Western wall of the bell-tower.

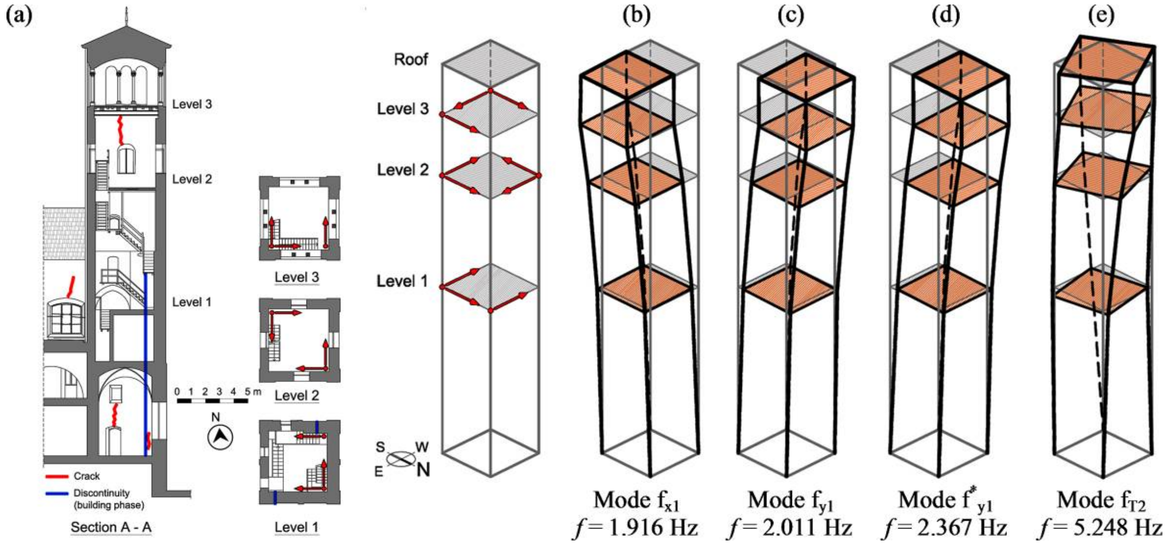

Figure 9. (a) Schematic of accelerometers layout and (b)-(e) selected vibration modes identified from ambient vibration tests.

The construction sequence adopted for the tower, not identified before, raised obvious concern about the performance of the structure under wind and seismic actions. Therefore, a wide research program was planned to assess the structural condition of the building and is currently in progress. In more details, the research consists of the following steps [14]: (a) prompt on-site investigation, including geometric survey and visual inspections; (b) static monitoring of the main cracks through the installation of 10 displacement transducers (as well as 5 temperature sensors) at different levels of the tower; (c) ambient vibration testing and identification of the dynamic characteristics of the tower; (d) installation of a simple dynamic monitoring system in the tower.

Ambient vibration tests were carried out on 23 September 2015 [14] and mainly aimed at evaluating the baseline dynamic characteristics of the tower before the installation of a continuous dynamic monitoring system in the building. Selected mode shapes identified by applying the SSI-Cov method are presented in Fig. 9 and reveal very peculiar dynamic characteristics of the tower, that are conceivably related to the structural arrangement and construction sequence of the building. In more details, closely spaced modes with similar mode shapes were clearly identified, so that the sequence of identified modes turns out to be very different from the expected regular series of two bending modes (one for each principal plane of the structure) and one torsion mode (see e.g., the *San Vittore* bell-tower). As shown in Fig. 9, the identified sequence of vibration modes (Fig. 9) includes: (a) the fundamental mode (f_{x1} = 1.92 Hz, Fig. 9b), involving dominant bending in the E-W direction; (b) two bending modes in the N-S direction, that are characterized by closely spaced natural frequencies (f_{y1} = 2.01 Hz and f^*_{y1} = 2.37 Hz) and very similar mode shapes (Figs. 9c and 9d).

The continuous dynamic monitoring system (Fig. 10a) installed in the tower since 22 October 2015 includes 4 MEMS accelerometers (Kistler model 8330A3, 1.2 V/g sensitivity, ± 3.00 g peak acceleration, 1.3 µg resolution and 0.4 µg/√Hz r.m.s. noise density), one Ethernet carrier with NI 9234 data acquisition module and one local PC for the management of the continuous acquisition and the data storage. In addition, 5 temperature sensors – denoted as T_{0N}, T_{1E}, T_{2E}, T_{2W} and T_S in Fig. 10a – were available, and measured both the indoor temperature at different levels of the tower and the outdoor temperature on the South side of the structure; hence, a relatively dense representation of the temperature conditions of the tower is achieved.

Figures 10b-c show the hourly averaged value of the measured temperatures in the monitoring period from 22 October 2015 to 21 October 2016. Table 3 summarizes the correlation coefficients between the environmental data during the same period. Figures 10b-c and Table 3 indicate that a large degree of correlation exists between all temperature data, with the measurements T_{1E}, T_{2E}, and T_{2W} being almost perfectly correlated and characterized by correlation coefficients very close to unity.

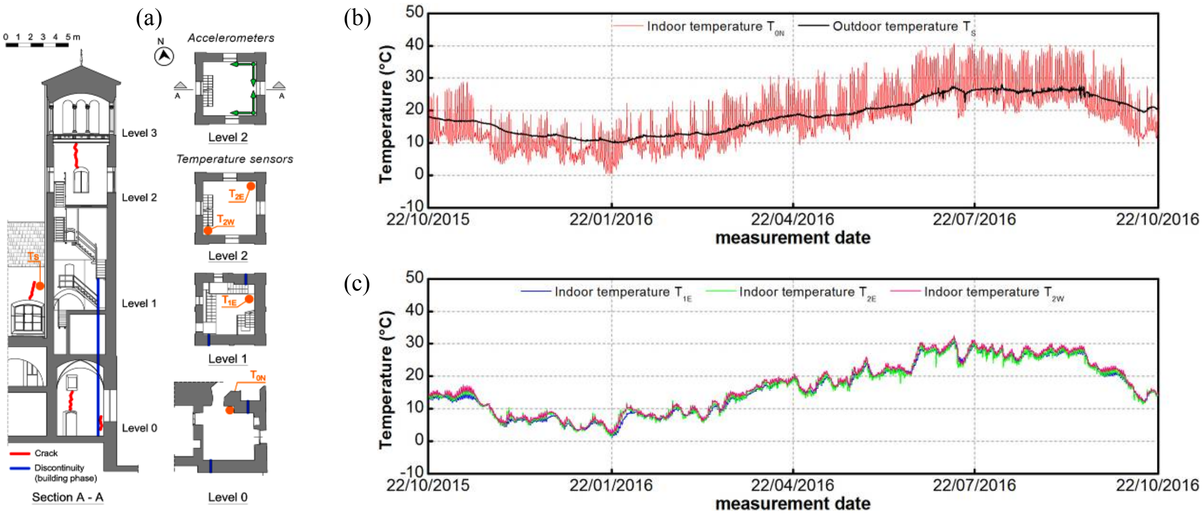

Figure 10. (a) Accelerometers and temperature sensors installed in the tower; (b) Variation in time of the measured temperatures T_{0N} and T_S; (c) Variation in time of the measured temperatures T_{1E}, T_{2E} and T_{2W}.

Table 3. Correlation coefficients between the measured temperatures (from 22/10/2015 to 21/10/2016).

	T_{0N}	T_{1E}	T_{2E}	T_{2W}	T_S
T_{0N}	1.000	0.967	0.957	0.960	0.837
T_{1E}		1.000	0.995	0.997	0.873
T_{2E}			1.000	0.998	0.900
T_{2W}				1.000	0.885
T_S					1.000

Figure 11 presents the evolution of the automatically identified modal frequencies in the first year of continuous dynamic monitoring (i.e., from 22/10/2015 to 21/10/2016), whereas the relevant statistics are summarized in Table 4 through the mean value (f_{ave}), the standard deviation (σ_f), and the extreme values (f_{min}, f_{max}) of each natural frequency. The results summarized in Fig. 11 and Table 4 allow the following comments:

1. Notwithstanding the low level of the ambient excitation, 4 normal modes were identified with high occurrence and accuracy;
2. The natural frequency of modes f_{x1} and f_{T2} exhibits significant increase in Spring and Summer period. This trend suggests that these modal frequencies are strongly affected by the temperature and − similarly to *Gabbia* tower and *San Vittore* bell-tower − increase with increased temperature;
3. On the contrary, the natural frequency of modes f_{y1} and f^*_{y1} exhibits very limited variation, with the standard deviation being equal to 0.009 and 0.015 Hz, respectively. For those modes, the frequency trend increase with increased temperature is conceivably balanced by the loss of tension in the metallic tie-rod placed on the West side and connecting the North and South load-bearing walls of the tower;
4. As shown in Fig. 11a, the natural frequencies of the two lower modes f_{x1} and f_{y1} exhibit crossing in Summer months. To the best of the authors' knowledge, this behavior has not been observed before on masonry towers and conceivably depends on the different effect exerted by the temperature on the natural frequencies of the two modes. Furthermore, the mode shape of both modes f_{x1} and f_{y1} tends to hybridize when crossing occurs: in other words, the two modes tend to involve biaxial bending in both the main E-W and N-S directions when the natural frequencies become very close to each other. It is further noticed that this hybridization, suggesting the occurrence of something similar to mode veering (see e.g. [24]), is more significantly detected for mode f_{y1}.

It is further noticed that ongoing numerical investigation indicates that the PCA-based regression turned out to be an effective tool to mitigate the environmental effects on automatically identified frequencies, in spite of the relatively small number of monitored frequencies.

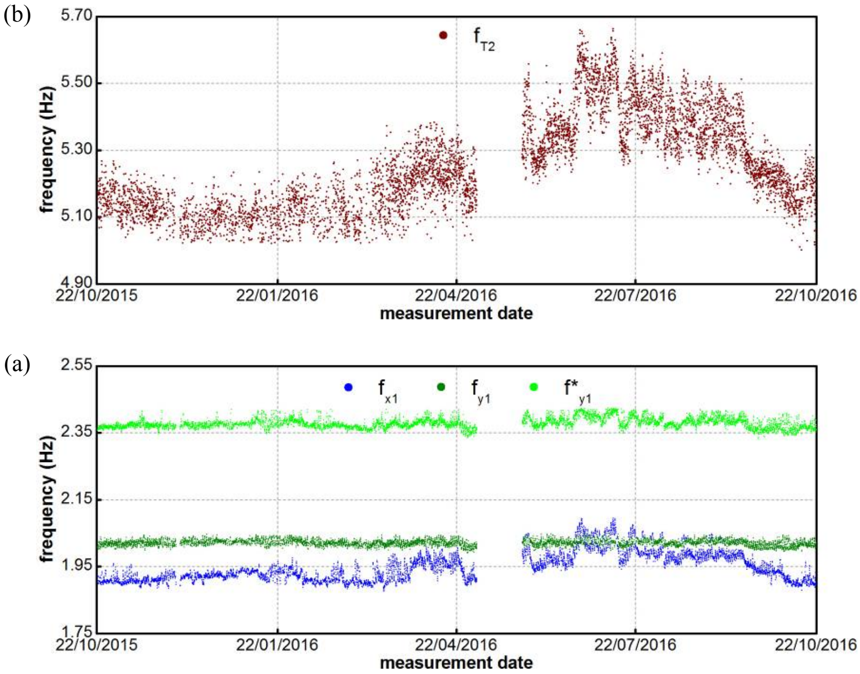

Figure 11. Time evolution of the automatically identified natural frequencies during the first year of monitoring (from 22/10/2015 to 21/10/2016): (a) modes f_{x1}, f_{y1} and f^*_{y1}; (b) mode f_{T2}.

Table 4. *Santa Maria del Carrobiolo* bell-tower: Statistics of the natural frequencies identified from 22/10/2015 to 21/10/2016.

Mode	f_{ave} (Hz)	σ_f (Hz)	f_{min} (Hz)	f_{max} (Hz)
f_{x1}	1.946	0.041	1.876	2.094
f_{y1}	2.020	0.009	1.990	2.053
f^*_{y1}	2.379	0.015	2.333	2.423
f_{T2}	5.265	0.141	5.001	5.663

3. CONTINUOUS MONITORING AND SHM OF COMPLEX HISTORIC BUILDINGS: THE CATHEDRAL OF MILAN

The Milan Cathedral (Figs. 12-13), erected between 1386 and 1813, is one of the largest masonry monuments ever built. The church exhibits the tallest main nave among Gothic Cathedrals, with the height of the vault intrados of the main nave being at about 45 m from the ground. The overall dimensions of the Latin cross-shaped plan are about 66 m × 158 m (Fig. 14), with the aisles and the central naves spanning 9.6 m and 19.2 m, respectively.

When compared with other Gothic cathedrals, the Milan Cathedral exhibits a peculiar structural system, with metallic tie-rods being permanently installed under each vault (Fig. 12b) and designed to exert an active part in resisting the lateral thrusts. Historical documents, dating back to year 1400, testify that the tension bars in the Milan Cathedral were permanently installed on the top of the piers during the construction with the aim of reducing the horizontal thrust on the lateral buttresses, as those buttresses were judged too slender by the French architect Jean Mignot. A total of 122 metallic tie-rods (Fig. 12b) is nowadays present in the Milan Cathedral and most of them are the original elements dating back to the age of construction.

(a)

(b)

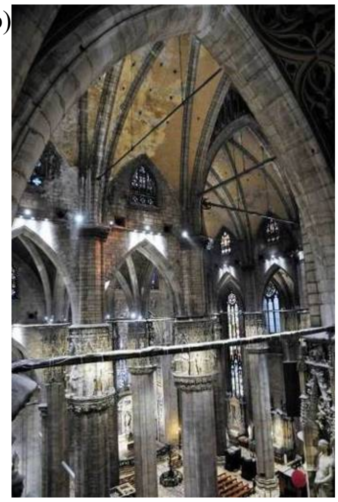

Figure 12. Milan Cathedral: (a) Aerial view (courtesy of Veneranda Fabbrica del Duomo di Milano); (b) Inside view of arches, vaults and iron tie-rods.

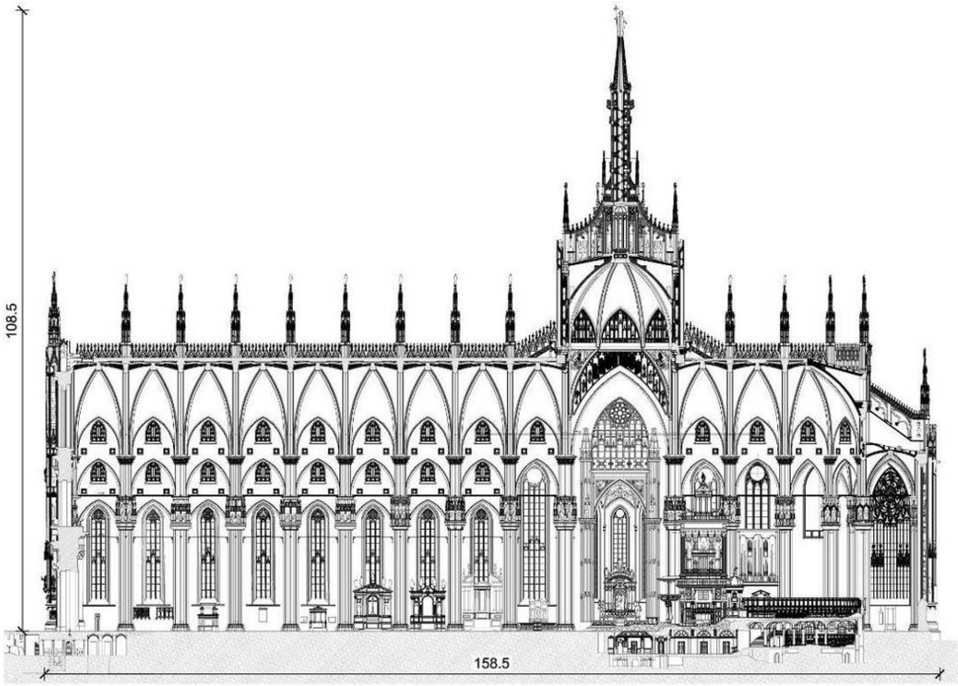

Figure 13. Longitudinal section of the Milan Cathedral (dimensions in m). (courtesy of Veneranda Fabbrica del Duomo di Milano)

Within the traditional collaboration between Politecnico di Milano and *Veneranda Fabbrica del Duomo di Milano* – the historic Institution established in 1387 and responsible for the preservation and development of the Cathedral – a structural monitoring system was recently designed and implemented with the two-fold objective of assisting the condition-based structural maintenance of the Cathedral and creating a large archive of experimental data useful to improve the knowledge of the monument.

The new monitoring system, fully computer based and with efficient transmission of the collected data, includes quasi-static and dynamic measurements [18]. The static monitoring system consists of: (a) bi-axial tilt-meters installed at the top of selected piers and at 3 levels of the Main Spire; (b) vibrating wire extensometers mounted on the iron tie-rods which are characterized by the higher tensile stress; (c) temperature and humidity sensors for the measurement of internal and external environmental parameters. The dynamic monitoring is performed through seismometers (electro-dynamic velocity sensors) installed at the top of 14 selected piers and at 3 levels of the Main Spire.

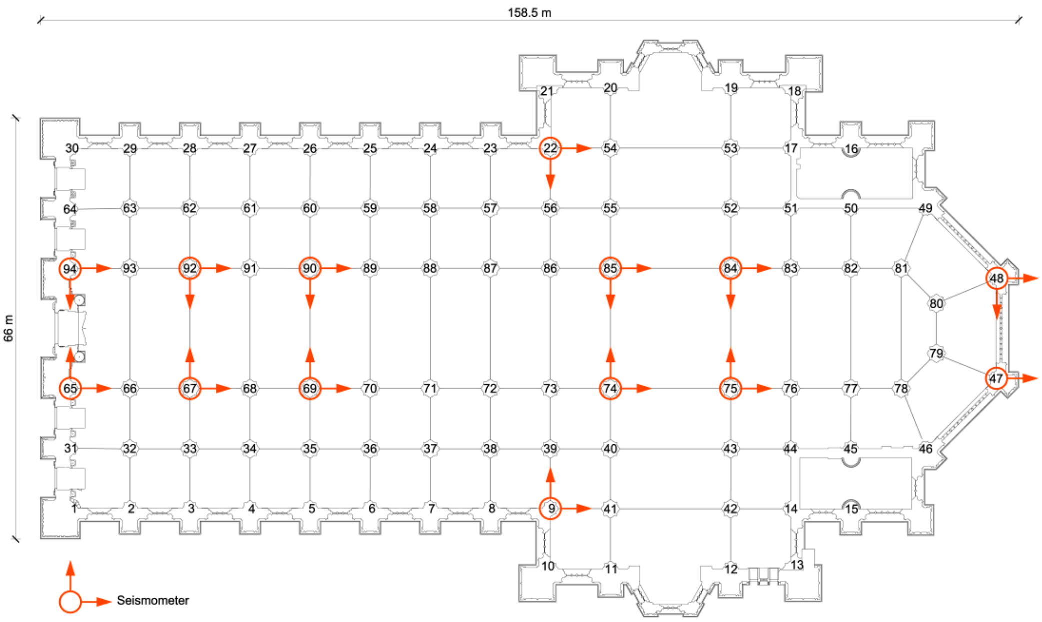

Figure 14. Map of the seismometers permanently installed inside the Milan Cathedral (dimensions in m).

3.1. The dynamic monitoring system

The dynamic monitoring system installed in the Cathedral of Milan is entirely based on SARA SS45 seismometers (electro-dynamic velocity transducers) [25]. The seismometer choice is motivated by: (a) the high sensitivity (78 V/[m/s]) and the excellent performance of electro-dynamic transducers in the low frequency range ($f \leq 100$ Hz); (b) the un-necessity of powering the sensors; (c) the possibility of obtaining a good estimate of the displacement time series by integrating the velocity records.

13 bi-axial seismometers and 1 mono-axial seismometer were installed at mounted at the top of selected piers (Fig. 14) and are continuously measuring the velocity in the two orthogonal N-S (transversal) and E-W (longitudinal) directions. The sensors installed on piers (94, 92, 90), (65, 67, 69), (22, 85, 84), (9, 74, 75) and (47, 48) are grouped and wired to five 24-bit digitizers SARA SL06 [25] and equipped with UMTS modems for data transfer. In addition, 9 seismometers are installed at 3 different levels of the Main Spire.

Preliminary dynamic tests were performed in the Milan Cathedral, by installing conventional accelerometers: (i) on the top of piers 22, 54 and 55 (6 channels of data, with 3+3 accelerometers oriented along N-S and S-W directions, respectively); (b) on the top of piers 9 and 39 (4 channels of data, with 2+2 sensors oriented along N-S and S-W directions, respectively). The results of these tests allowed to clearly identify 3 global modes of the Cathedral [18]. The lower vibration modes involve global motion of the Cathedral in the direction N-S (1.39 Hz) and E-W (1.70 Hz), whereas a certain classification of third mode (2.65 Hz) was not possible.

3.2. Dynamic characteristics of the church and SHM strategy

The dynamic monitoring system has been active since 16/10/2018. The automated OMA allows to identify 8 normal modes in frequency range 0-5 Hz. Figure 15 shows the lower 6 modes, as identified at the beginning of the monitoring (17/10/2018, h 11:00-12:00). It should be noticed that: (a) the lower two modes, as expected, are global sway modes of the Cathedral along the transverse (N-S, 1.39 Hz, Fig. 15a) and longitudinal (E-W, 1.70 Hz, Fig. 15b) direction; (b) the subsequent two modes (1.96 and 2.48 Hz, Figs. 15c and 15d) involve dominant motion in the N-S direction and bending of the naves; (c) the 5th mode involves out-of-phase bending of the North and South naves (2.64 Hz, Fig. 15e); (d) the 6th mode involves out-of-phase motion of the façade and apse in the E-W direction (2.76 Hz, Fig. 15f); (d) the damping ratios of all modes are quite high and range between 1.91% and 5.58%; (f) the longitudinal modes (E-W, Figs. 15b and 15f) seem to exhibit higher damping ratios.

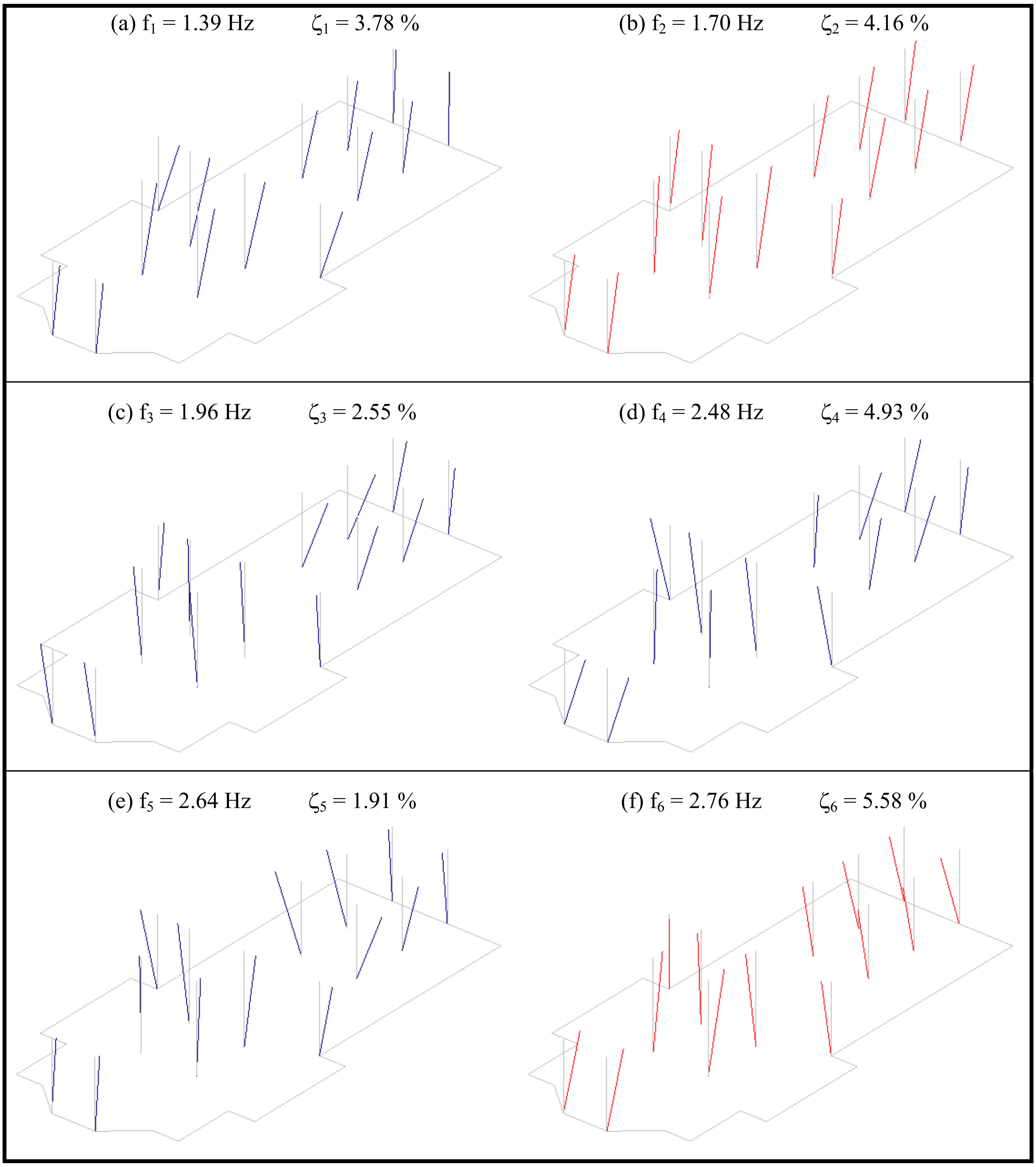

Figure 15. Selected vibration modes of the Milan Cathedral.
(The blue and red color highlights N-S and E-W dominant motion, respectively)

Figure 16a shows the evolution in time of the average indoor temperature during the first 4 months of monitoring. The corresponding variation of the modal frequencies versus time is illustrated in Fig. 16b. Even if the monitoring period is still quite limited, the dependence of resonant frequencies on (both indoor and outdoor) temperature seems to be very different from the one observed in masonry towers or in other masonry curches [10], [12-13]: the natural frequencies tend to increase with decreased temperature. Hence, during Winter time, the dependence of frequencies on temperature is conceivably driven by the iron tie-rods installed in the Cathedral (Fig. 12b).

It should be noticed that – unlike previous monitoring of churches [4], [10], [12-13] and palaces [19], where a limited number of quasi-static and dynamic sensors were permanently available – the Cathedral of Milan has been instrumented with a relatively large number of sensors [18] and also the environmental parameters are extensively monitored.

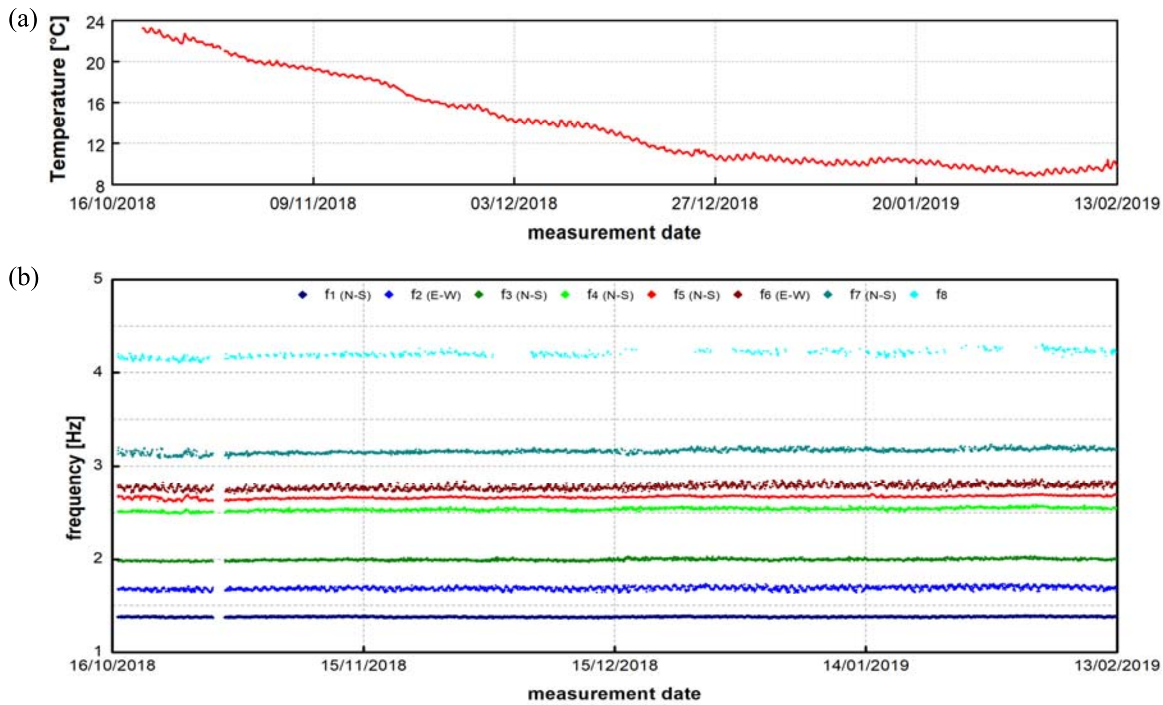

Figure 16. Time evolution (from 16/10/2018 to 12/02/2019) of: (a) the average temperature inside the Cathedral (b) the automatically identified natural frequencies..

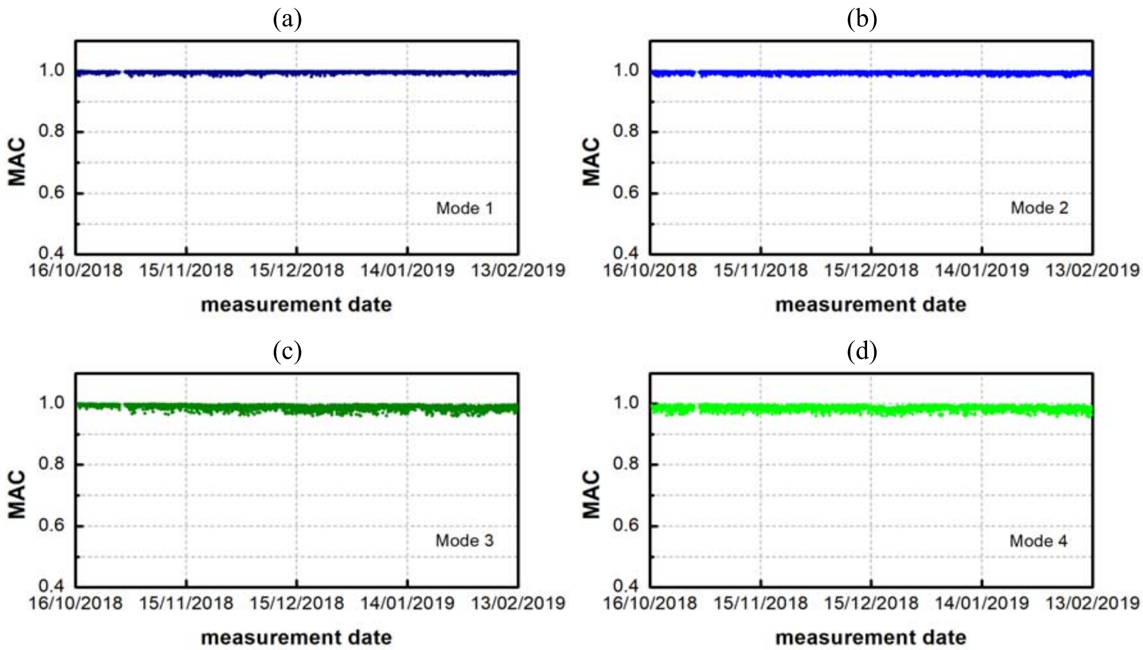

Figure 17. Time evolution (from 16/10/2018 to 12/02/2019) of the MAC of selected modes (with respect to the beginning of the dynamic monitoring).

Consequentlly, the SHM of the historic monument will be performed not only using the resonant frequencies but also checking the invariance of the mode shapes (Fig. 17) and related modal complexity [20]. In addition, the availability of quasi-static measurement of structural responses (such as the tilt of selected piers and the strain on a certain number of tie-rods) might contribute – directly or through the fusion with the data coming from OMA-based monitoring – to check the structural health condition of the Cathedral.

4. CONCLUSIONS

Selected results of the continuous dynamic monitoring of three historic towers have been reported in the first part of paper with the objectives of: (a) demonstrating that the successful monitoring of the dynamic characteristics can be obtained by permanently installing a few high-sensitivity accelerometers (or seismometers) in the upper part of the building; (b) better understanding the effects of changing temperature on the time evolution of continuously identified modal frequencies and (c) highlighting that the occurrence of slight damage under changing environment can be identified even installing a limited number of sensors in the structure.

Different temperature-driven effects on frequency changes have been observed in the investigated towers: (1) the increase of natural frequencies with increased temperature, due to the closure of superficial cracks and minor masonry discontinuities induced by the thermal expansion of materials; (2) the quick and significant increase of the modal frequencies with decreased temperature around the freezing condition (conceivably induced by the presence of ice, which fills and closes the cracks and causes a temporary stiffening of the structure); (3) the effects of the increased temperature at the local level, such as the increase of thrust exerted by inclined structural elements or the slackening of metallic ties.

In the second part of the paper, the focus is moved to the vibration monitoring of complex Cultural Heritage buildings, such as the Cathedral of Milan, where a relatively large monitoring system has been recently designed and implemented. In such instances, the possibility of adopting a SHM strategy which involves all the available modal parameters (i.e., resonant frequencies, mode shapes and modal complexity) has been discussed.

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