Pre-diagnostic prompt investigation and static monitoring of a historic bell-tower

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An extensive research program is currently in progress to assess the structural condition of the bell-tower of the Church *Santa Maria del Carrobiolo* in Monza, Italy and to address the preservation of the historic building. The research program was consequent to the direct survey of the tower, carried out within a wide cataloguing activity of the main religious buildings in Monza and highlighting a weak structural lay-out of the bell-tower. The paper presents the main results of the investigation program performed to date and including: (a) documentary research, systematic visual inspection on site and experimental evalua-tion of the fundamental period of the tower, carried out using an industrially engineered microwave interferometer; (b) installation of a static monitoring system aimed at surveying the opening of the main cracks, possibly related to the recent construction of an underground car park in the close neighbourhood of the church; (c) dynamic tests in operational conditions, performed using conventional high-sensitivity accelerometers with the twofold objective of validating a FE model of the tower and implementing the installation of a continuous dynamic monitoring system.

HIGHLIGHTS

- Visual inspection revealing a weak structural layout of a masonry bell-tower.
- Evaluation of the fundamental period using microwave remote sensing.
- Installation of a static monitoring system in the bell-tower.
- Evaluation of the dynamic characteristics of the historic tower using OMA.
- Preliminary assessment of the environmental effects on lower natural frequencies.

1. Introduction

The technical challenges of historic building preservation involve evaluating possible vulnerabilities as well as addressing damage mitigation and repair actions within a strategy of minimal intervention and maintenance. These objectives require deep knowledge of the materials and structure, the local and global state of preservation, the potential occurrence of damage and its causes. In order to fill the gap between the initial low knowledge and the complexity of historic structures, experimental investigations – including inspections, surveys, laboratory tests on sampled materials and on site tests – are required and recommended by Codes of Standards in several countries [1–5].

Critical issues to be addressed in the structural assessment of the historic structures are: (a) the lack of knowledge about the construction techniques, the structural changes over time and the effects of the decay and local damages (even if mapped in detail); (b) the criteria and the procedures to get these information, (c) merging the pieces of information obtained from several methodologies and disciplines in a diagnostic process [6,7]. For instance, the masonry quality is highly dependent on manufacturing practice and decay, resulting in significant variation of the mechanic characteristics that might be investigated through limited opportunities of sampling and testing. In addition, the discussion on the correlation between the results of local tests and quantitative parameters needed to build up global structural capacity models is still open. Furthermore, the structural model of a historic structure, even when based on accurate field survey, always involves simplified assumptions and several uncertainties in the material properties and their distribution, in the geometric layout and in

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the boundary conditions [8]. Other uncertainties are related to the structure evolution and the modelling of damages and subsequent repairs [7].

In conclusion, the structural assessment of historic structures is a complex and articulated procedure, which could require time-consuming (and expensive) investigations in order to examine in depth several issues. Unfortunately, wide investigation programs are seldom possible, due to economic issues or to emergency conditions (such as the post-earthquake assessment).

The paper firstly presents the procedures adopted and the main results obtained in the prompt assessment carried out on the bell-tower of the Church *Santa Maria del Carrobiolo* in Monza, Italy [9,10]. Subsequently, a wider research program was planned and is currently in progress. In more details, the research consists of 5 successive steps:

(1) Prompt on-site investigation [11], including geometric survey, visual inspections and evaluation of the tower's fundamental period using microwave remote sensing [12–14], aimed at defining the main issues.
(2) Static monitoring of the main cracks.
(3) Ambient vibration testing and complete dynamic characterisation of the tower using Operational Modal Analysis (OMA, see e.g. [15]).
(4) Development of a FE model of the tower and vibration-based structural identification of the uncertain parameters of the model [16,17].
(5) Design and installation of a continuous dynamic monitoring system in the tower [18].

As previously pointed out, within a wide cataloguing program of several religious buildings in Monza, a pre-diagnostic survey of the *Santa Maria del Carrobiolo* bell-tower (Fig. 1) was initially performed. The survey – partially supported by the CARIPLO Foundation and aimed at collecting prompt information on the historic buildings – included topographic survey, visual inspection, documentary research and evaluation of the fundamental period. Direct survey indicated that the tower was built after the church and revealed that two sides of the bell-tower are directly supported by the load-bearing walls of the church apse and right (South) aisle. The construction sequence adopted for the tower, not identified before, raised obvious concern about the performance of the structure under normal and exceptional loads. In addition, the recent construction of an under-ground car park adjacent to the East side of the tower conceivably activated movement of the pre-existing cracks and of the structural discontinuities related to construction phases.

After a brief description of the planned research program and of the investigated bell-tower, the paper fully details the results of the steps (1)–(3) outlined before and emphasis is given on the principles adopted, the importance of the collected information within the diagnostic context and the choice of installing a continuous dynamic monitoring system in the bell-tower.

2. Investigation procedures

The methodology adopted to currently evaluate and to continuously assess the structural condition of the bell-tower at study (calibrated through the progressive refinement of diagnostic procedures [11,16–18]) is based on: (a) geometric survey and local inspections supported by stratigraphic analysis, (b) ambient vibration testing; (c) development of a dynamics-based FE model; (d) static and dynamic monitoring.

On-site inspections are fundamental to collect information on the geometry, the presence of local damage and its extension, and to identify the regions where more accurate observations (e.g. crack monitoring) have to be concentrated [6]. The crack pattern and the masonry discontinuities should be accurately classified and documented by pictures and reported on the geometrical survey.

Concurrently, the historical evolution of the structure has to be investigated through historic and documentary research in order to explain the signs of damage or the irregularities detected on the building. Despite the importance of the historic and documentary research, it may require long time to get exploitable information so that, in emergency conditions, the evaluation of the building evolution has to be initially performed through the analysis of the masonry textures and the available references. Of course, the direct observations should be supplemented at a later time by the refinement and conclusion of the historic and documentary research.

Since the first approaches to the investigated building, the inspection must be addressed to detect anomalies which could affect the structural behaviour. Systematic attention must be paid to the corner layout, the crack pattern, the discontinuities but also to any change of the masonry textures by stratigraphic methods. The latter information is closely related to the construction phases: the quality of the link between each homogeneous masonry por-
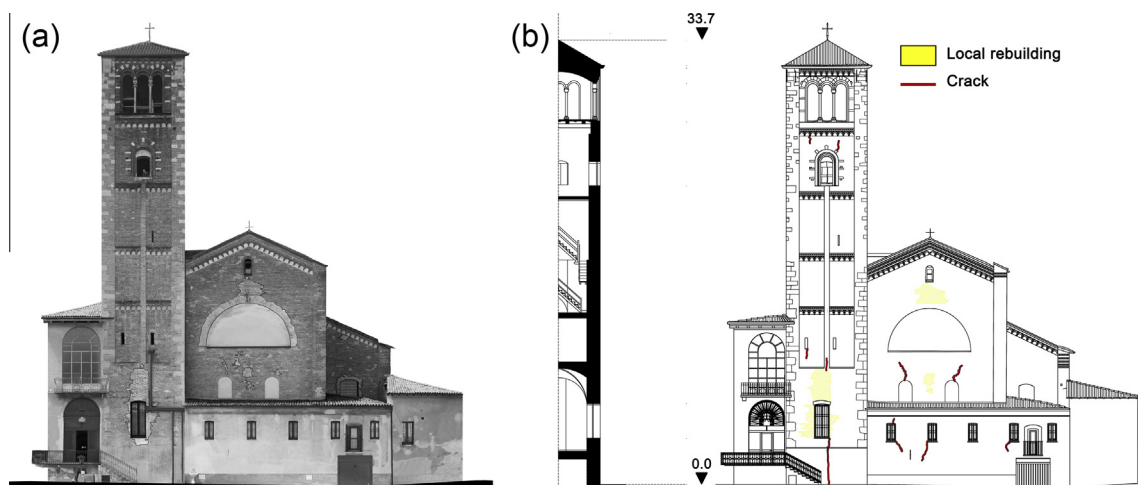

Fig. 1. The Church of *Santa Maria del Carrobiolo* and the bell-tower: (a) Orthoplane; (b) Geometric survey of the East front of the tower and the church apse (cracks and local rebuilding are mapped, dimensions in m).

tion highly affects the static/dynamic behaviour but also the damage triggering. It is indeed very common the construction of new portions of the building or opening infilling without any link to the former masonry structure. Furthermore, the coupling of ineffective links between the building phases and the extension of masonry decay or damage can produces local vulnerability [2,5,11,18] to be better investigated.

Just after the visual survey, ambient vibration tests should be carried out, measuring the dynamic response of the structure to ambient excitation and extracting the modal parameters (i.e. natural frequencies, mode shapes and damping ratios) from the measured data. The identified modal parameters, describing the structure global behaviour in terms of characteristics of the key vibration modes, could be used both to validate FE models [16,17,19–23] and as features sensitive to the structural condition in long-term monitoring [18,24]. It is worth mentioning that, in continuous monitoring, the relationships between the features assumed as sensitive to the structural condition and the environmental parameters (e.g. temperature) need to be investigated for early detection of abnormal structural changes: once the environmental effects have been explored and can be filtered out to normalize response data, any further changes in the sensitive features should rely to changes of the structural condition.

If the ambient vibration test is performed in emergency conditions, non-contact measurement techniques can be used [12–14]. For instance, microwave remote sensing might be extensively applied for identifying the dynamic characteristics of masonry towers when: (1) the fundamental mode is mainly needed (as it happens in wide programs of preventive conservation or in post-earthquake assessment); (2) special issues have to be investigated (such as the effects of bell swinging). In those cases, the use of radar-based remote sensing is very quick and accurate and does not expose the test crew to hazardous conditions.

3. Description of the bell-tower and prompt on-site tests

The complex of *Santa Maria del Carrobiolo* includes a church, a monastery, a bell-tower, an oratory and other minor buildings, which were erected at different times. The complex was originally built by the Italian religious order known as Humiliati – probably formed in the XII century and suppressed in 1571 – and passed to the Barnabite order in 1574. Based on the historic documents, the construction of the church and the monastery dates back to XIII century [9], whereas a date engraved on a stone of the bell-tower traces the likely conclusion of its construction back to 1339 [9].

The tower (Fig. 1), about 33.7 m high, is built in solid brick masonry and has nearly square plan (5.93 m × 5.70 m); the average thickness of the load bearing walls slightly decreases from 70 cm at the ground level to 62 cm at the top.

The direct survey of the structural discontinuities provided clear evidence of the construction sequence and confirmed that the bell-tower was built after the erection of the apse and right aisle of the church. In more details, the inspection of the structural discontinuities revealed that: (a) two sides of the tower are directly supported by the load-bearing walls of the church apse and the last part of right aisle (Figs. 2 and 3); (b) from the base up to about the height of the church, the load-bearing walls of the tower (East and South side) were just leant against (but not toothed or linked to) the existing walls of the church; (c) the discontinuities due to the building phases look very sharp, again demonstrating the lack of connection between the portions (Figs. 4 and 5). It is further noticed that the same brick decoration placed on the outer pilaster of the church apse is nowadays embedded in the tower walls (Fig. 4).

Above the church height, the masonry texture seems homogeneous, even if several cracks are present (Fig. 5). Major cracks are visible indoor in the centre of both East and West fronts of the tower, whereas in the centre of the outdoor fronts the cracking state is partially hidden by pilasters (Figs. 4 and 5). Other meaningful cracks are concentrated at the level below the belfry and diffused on all the sides, across the large windows, and most of those cracks are cutting the entire wall thickness. Furthermore, a r.c. plain roof was built in the first decade of XX century on a portion of the South aisle and is partially supported by the West wall of the tower.

The recent construction of an underground car park, adjacent to the complex of *Santa Maria del Carrobiolo* on the East side, may have contributed to damage the tower and to activate the movement of pre-existing cracks and sharp discontinuities related to the construction phases. At the base of the tower, several cracks were observed in the plaster even after a recent wall painting.

The spread of cracks and the evident re-opening of cracks already repaired suggested the static monitoring of the main cracks, as it will be shown in Section 4.

In the context of the pre-diagnostic survey, dynamic tests in operational conditions were carried out to estimate the fundamental period of the tower and an innovative radar vibrometer [12–14], was used for the remote measurement of deflection time series of the bell-tower. Details concerning the theoretic background and the general testing procedures can be found in [13,14].

The radar sensor used in this work (Fig. 6) is an industrially engineered microwave interferometer (IDS, model IBIS-S) and consists of a sensor module, a control PC and a power supply unit. The sensor module is a coherent radar (i.e., a radar preserving the phase information of the received signal) generating, transmitting and receiving the electromagnetic signals to be processed in order to provide the deflection measurements. The sensor, including two horn antennas, has a weight of 12 kg and is installed on a tripod equipped with a rotating head, so that it can be aligned in any desired direction (Fig. 6).

The acquisition of the tower dynamic response was carried out in the E-W plane, perpendicular to the church apse and to the tower front toward the underground car park (Figs. 6 and 7a).
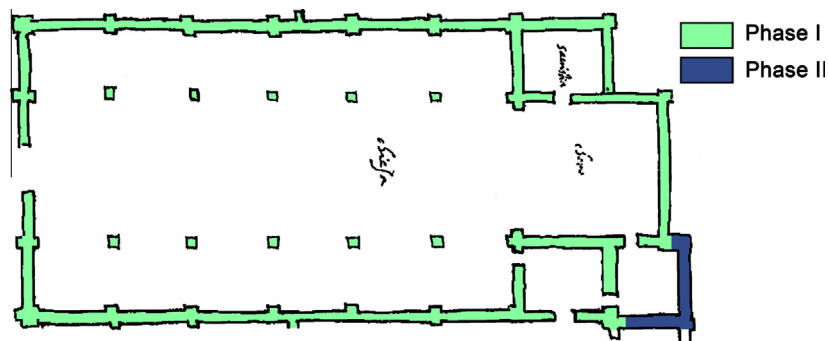

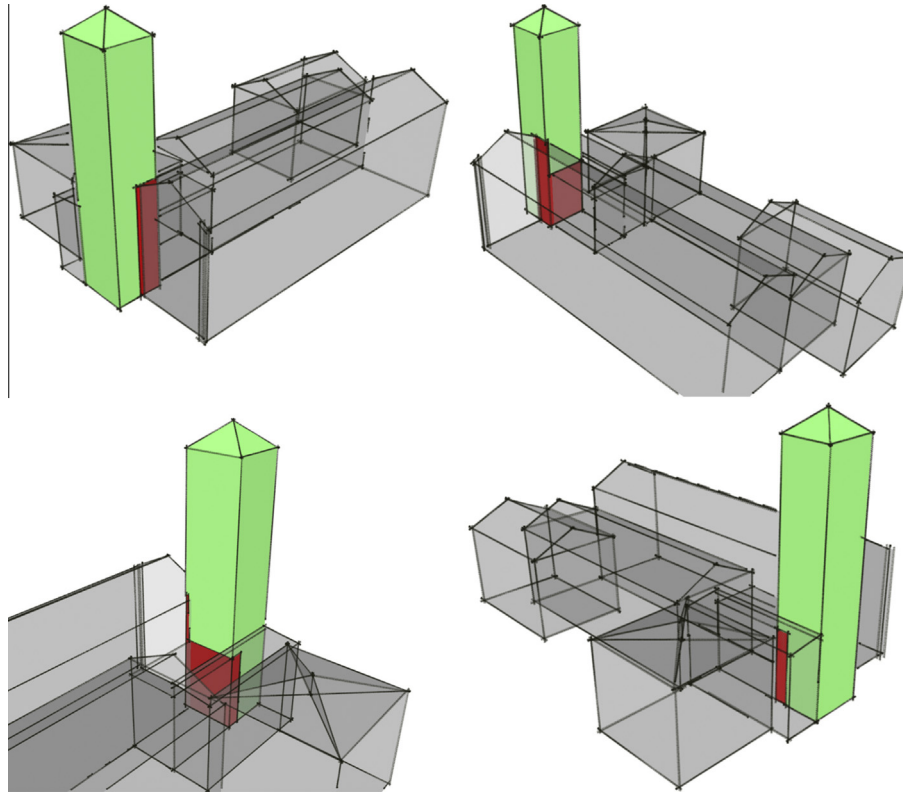**Fig. 2.** Building phases in a plan dating back to 1572.

Fig. 3. Schematic representation of the interaction between the church apse and the bell-tower.
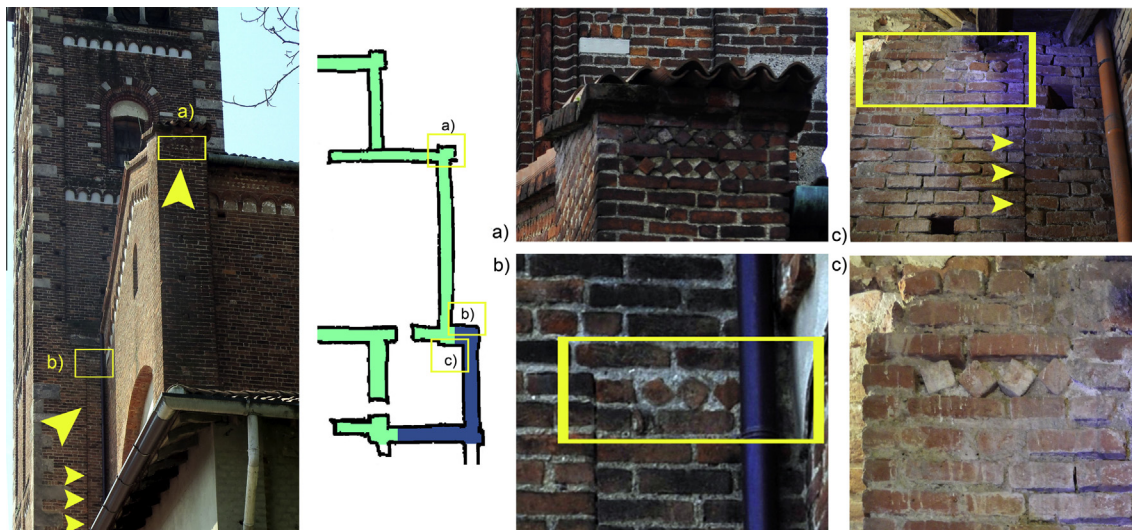

Fig. 4. Position of the decoration dating back to the first building phase and structural discontinuities.

Two ambient vibration tests were performed: in the first one, the excitation was only provided by micro-tremors and wind, whereas in the second test a higher level of excitation was induced by "random" swinging of bells. The tower responses were recorded in less than 2 h of a cloudy day, with the temperature changes being practically negligible.

The displacement response induced by micro-tremors at the top of the bell-tower turned out to be in the range of ±0.1 mm, whereas the response associated to the random swinging of bells was 4 times larger.

The frequency of the first vibration mode of the tower in the E-W direction (Figs. 7b and 8a), identified under pure ambient excitation (micro-tremors and wind), is equal to 1.93 Hz. The comparison between the first Singular Value (SV) line [25] of the spectral matrices of the datasets acquired under ambient excitation and random swinging of bells (Fig. 8), reveals slight but clear decrease – from about 1.93 Hz to 1.88 Hz – of the fundamental frequency when the structure is excited by the bell swinging. Hence, the dynamic characteristics of the tower are possibly dependent on the amplitude of excitation/response [26].
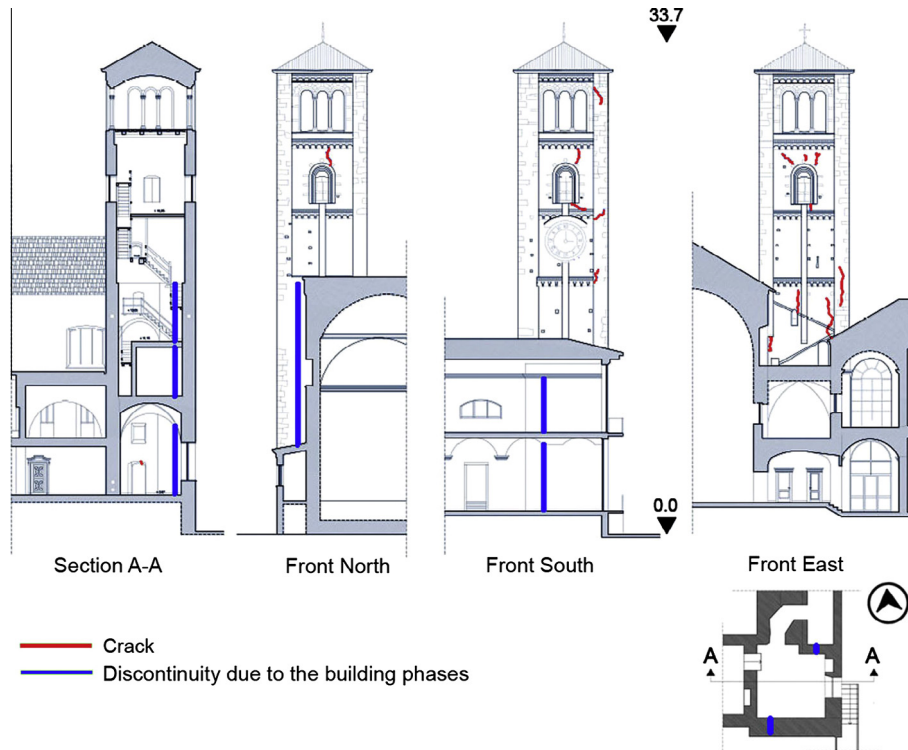
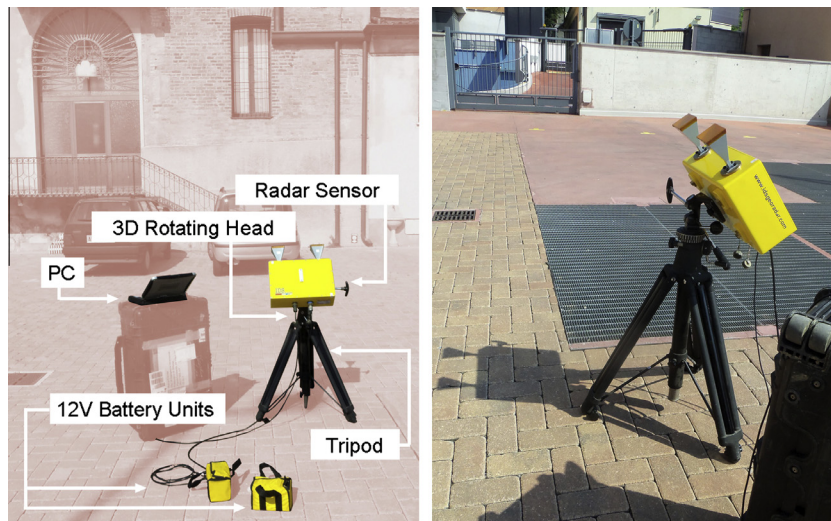**Fig. 5.** Section and fronts of the bell-tower.

Fig. 6. Views of the microwave interferometer (IDS, model IBIS-S).

4. Second phase: static monitoring

The recent re-opening of the repaired cracks suggested the installation of a static monitoring system in the tower. Hence, a network of 10 displacement transducers, integrated by 5 temperature sensors, was installed in the building to monitor the opening variations of the main cracks. Fig. 9 shows the general layout of the static monitoring system, installed on June 2014 and still active. It is worth mentioning that: (a) each couple of displacement transducers, along with one temperature sensor, is connected to a wireless data logger for the automatic data acquisition, storage and transmission by GSM-GPRS modem; (b) the automatic acquisition has been set to record the displacements and the temperatures every 10′; (c) the displacement transducers denoted as 1–2, 3–4 and 5–8 in Fig. 9 are placed inside the tower in order to check the opening of the main cracks at Level 0, Level 1 and Level 2, respectively; (d) the last couple of sensors (9–10 in Fig. 9) is installed on the outer wall of the church in the close neighbourhood of the tower.

The displacement transducers (Fig. 10) are linear sensors (potentiometers) with a maximum stroke of 25 mm and a maximum error on the linearity of 0.2%.

It is further noticed that the 5 thermocouples belonging to the static monitoring system allow to measure both the indoor temperature at different levels of the tower and the outdoor temperature on the South side of the structure (Fig. 9).
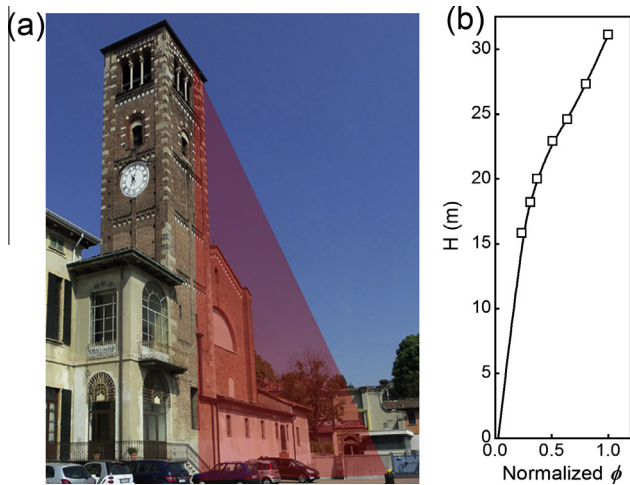
Fig. 7. (a) Radar position in the dynamic test of the tower; (b) Mode shape associated to the fundament frequency (f = 1.93 Hz, ambient excitation).
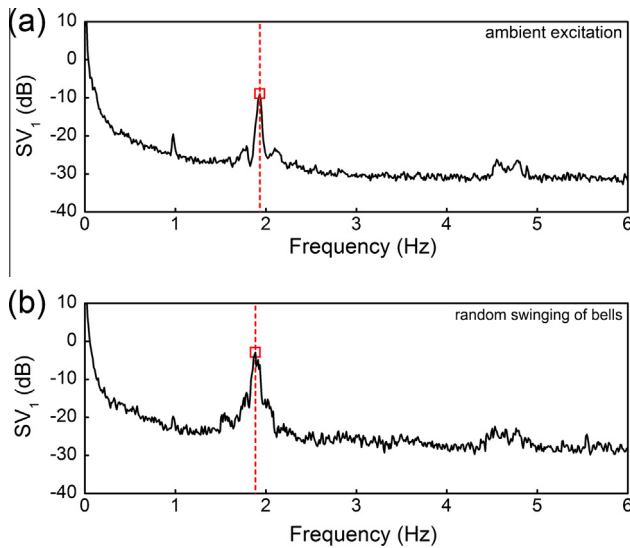

Fig. 8. First Singular Value (SV_1) line and identification of the fundamental frequency from radar data: (a) micro-tremors and wind; (b) random swinging of bells.

Although the importance of thermal effects in masonry towers has been only rarely investigated through numerical simulations [27], the dependence of the crack opening from the temperature is a well known phenomenon. This might conceal the damage effect on data, as any variation due to structural changes would be masked by the fluctuations caused by environmental factors. In fact, the measured deformations are actually the overlapping of components due to the structural behaviour and to changing environment.

In the present application, after 20 months of monitoring, the thermal effects seem dominant for the main cracks: one of the limits of the crack displacement monitoring is related to the type of data, too much local and requiring long time to distinguish structural issues from the environmental effects.

The most significant opening affects the cracks at ground level (Fig. 11a) but the thermal effects turned out to be largely dominant, so that possible settlements at the foundation level should have been conceivably very small. As shown in Fig. 11a and b, the evolution in time of measured temperature and displacement

exhibits almost regular trend, with an inverse correlation between temperature and displacement and periodic repetition of the displacements measured in similar temperature conditions.

As it has to be expected, all the main cracks tend to close with increased temperature (Fig. 11a and b), with the exception of the one denoted as TL2_SOCH2 (Fig. 9) and illustrated in the picture of Fig. 12a; Fig. 12b highlights that the opening of crack TL2_SOCH2 increases with increased temperature. This unexpected trend is conceivably associated to the structural effect exerted by the metallic tie-rod (Fig. 12a) placed above the crack and connecting the North and South load-bearing walls: as the temperature increases, the loss of tension in the tie-rod induces the opening of the cracks. It is worth mentioning that (as it will be discussed in Section 5) this behaviour also affects the evolution in time of one modal frequency of the bell-tower.

5. Third phase: ambient vibration tests

The results of the pre-diagnostic survey, including the prompt dynamic tests described in Section 3, suggested further investigation. Hence, two ambient vibration tests (AVTs) were carried out more recently by using a traditional data acquisition system with high-sensitivity accelerometers. Those AVTs were aimed at obtaining complete and accurate estimates of the dynamic characteristics of the tower with the twofold objective of validating a FE model of the structure [16,17] and implementing the installation of a continuous dynamic monitoring system in the building [18,24].

Dynamic tests were performed on 23/09/2015 and the response of the tower was measured at Levels 1, 2 and 3 of Fig. 9, according to the sensors layout schematically illustrated in Fig. 13. High sensitivity accelerometers (WR model 731A, 10 V/g sensitivity and ±0.50 g peak) measured the structural responses to ambient excitation at a sampling frequency of 200 Hz; the accelerometers were connected to a 24-channel data acquisition system (24-bit resolution, 102 dB dynamic range and anti-aliasing filters) through two-conductor cables.

Since the sampling frequency was much higher than that required for the investigated tower, low pass filtering and decimation were applied to the data before the modal identification. More specifically, the acceleration time series were low-pass filtered, using a 7th order Butterworth filter with cut-off frequency of 12.5 Hz, and decimated 8 times, reducing the sampling frequency from 200 Hz to 25 Hz.

The modal identification was performed using time windows of 3600 s and applying the covariance driven Stochastic Subspace Identification method (SSI-Cov [28]); the natural frequency estimates have been verified also by inspecting the first SV line of the spectral matrix, which is the mode indication function adopted in the Frequency Domain Decomposition (FDD [25]). The OMA results in terms of natural frequencies are summarized in Fig. 14, showing the first 3 SV lines of the spectral matrix and the stabilization diagram obtained by applying the FDD and the SSI-Cov, respectively. The inspection of Fig. 14 clearly highlights that the alignments of the stable poles in the stabilization diagram of the SSI 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.

The identified mode shapes are presented in Fig. 15 and reveal very peculiar dynamic characteristics of the bell-tower, that are conceivably related to its structural layout (i.e., the construction phases). The inspection of Fig. 15 reveals that the sequence of identified mode shapes is very different from those obtained in past experimental studies of similar towers [16–20,23,24], usually involving of a regular succession of two bending modes – one for each principal plane of the structure – and one torsion mode. On
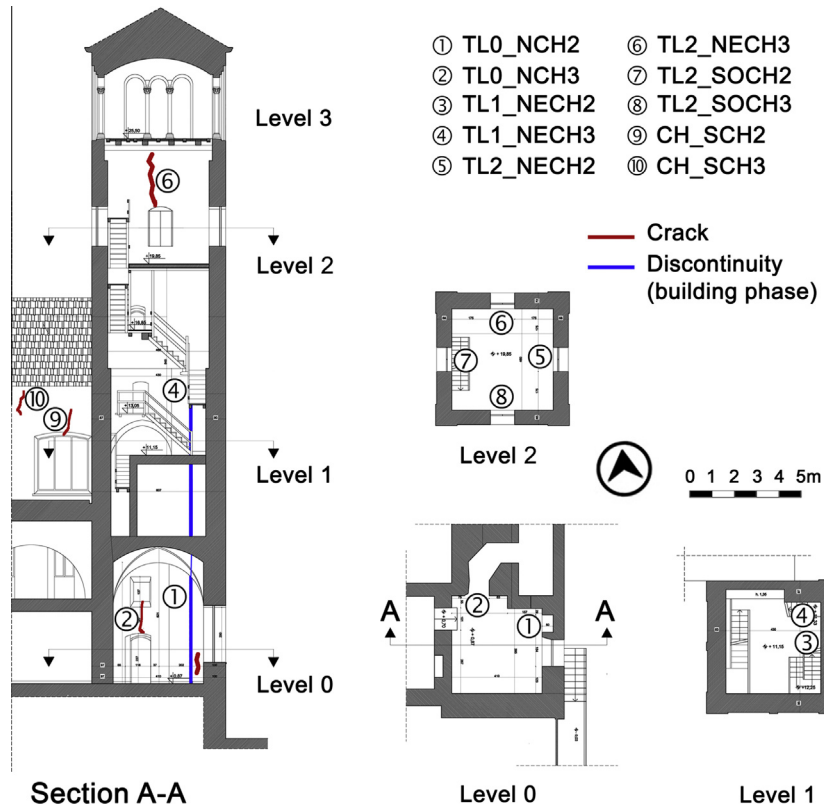
① TL0_NCH2   ⑥ TL2_NECH3
② TL0_NCH3   ⑦ TL2_SOCH2
③ TL1_NECH2   ⑧ TL2_SOCH3
④ TL1_NECH3   ⑨ CH_SCH2
⑤ TL2_NECH2   ⑩ CH_SCH3

— Crack
— Discontinuity (building phase)

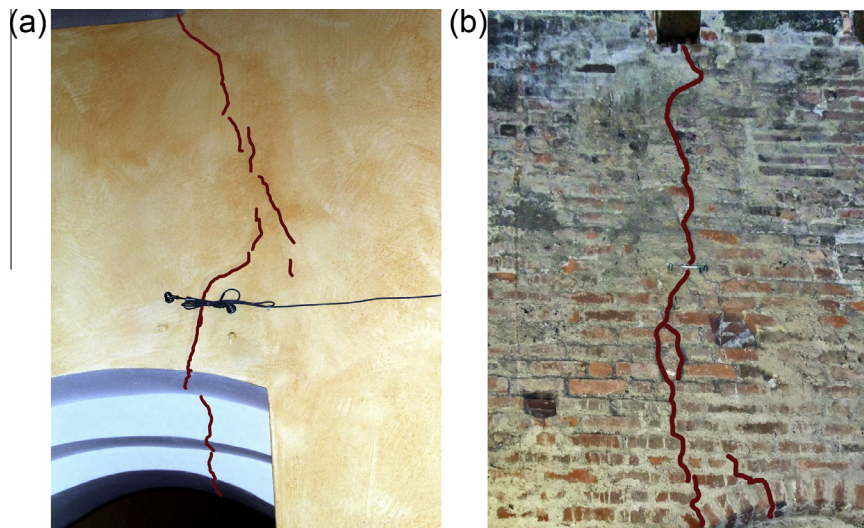**Fig. 9.** General layout of the static monitoring system.

Fig. 10. Displacement transducer (a) TL0_CH2 at Level 0 and (b) TL2_NECH3 at Level 2.

the contrary, the sequence of mode shapes of the investigated tower (Fig. 15) involves:

(a) One bending mode in the E-W direction (i.e., the fundamental mode already identified by using the radar interferometer).
(b) Two bending modes in the N-S direction (Fig. 15b and c), that are characterized by closely spaced frequencies and very similar mode shapes.
(c) Another mode of dominant bending, again in the N-S plane (Fig. 15d).

(d) Two torsion modes (Fig. 15e and f) with very similar mode shapes.
(e) The second bending mode (Fig. 15g) in the E-W direction.

Since the sequence of the identified modes and especially the two closely spaced modes involving bending in the N-S plane (Fig. 15b and c) turned out to be very unusual, a second series of AVTs was performed by installing 4 accelerometers at the Level 2 (Fig. 9) and continuously collecting the dynamic response of the tower for about one week. It should be noticed that the instrumented level – although not optimal for the identification of all
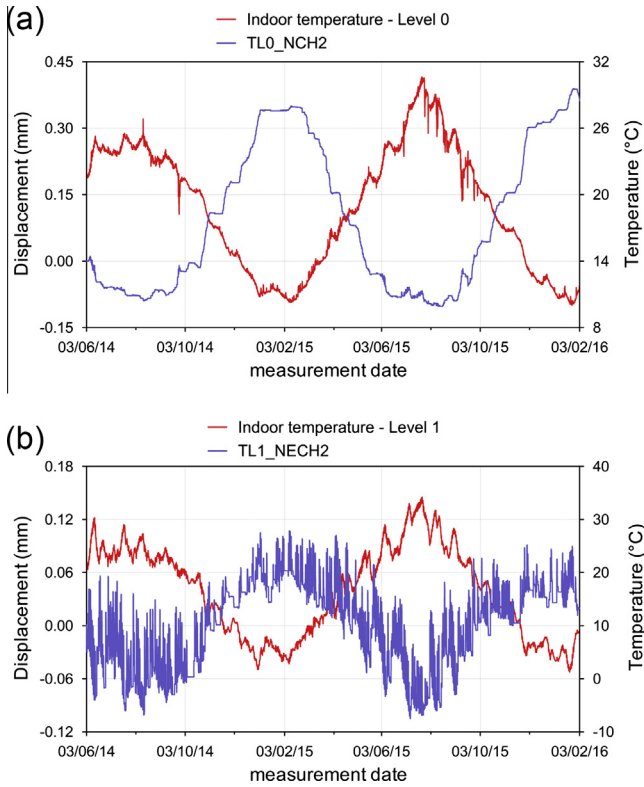
Fig. 11. Displacement and temperature measured in 20 months at: (a) Level 0 and (b) Level 1.
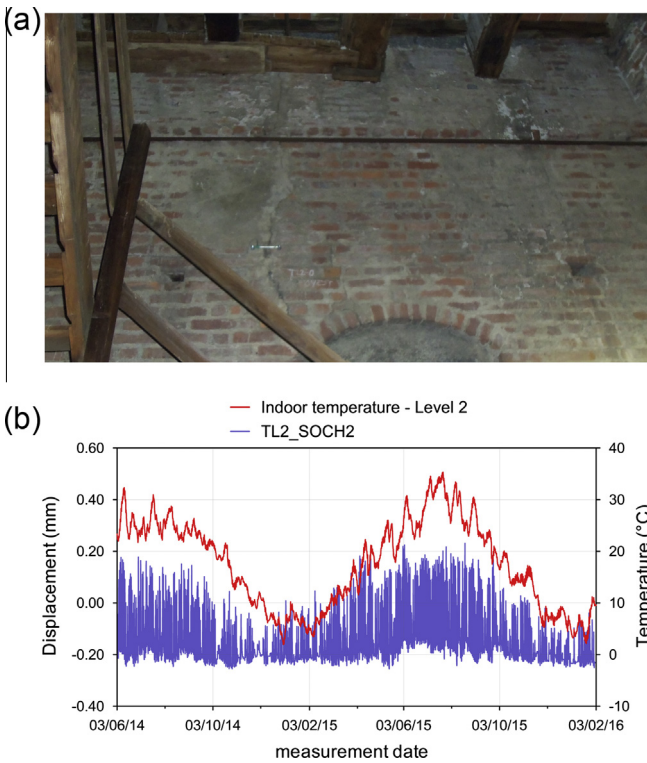

Fig. 12. (a) View of the transducer TL2_SOCH2 and the metallic tie-rod connecting the North and South load-bearing walls. (b) Indoor temperature and displacement measured by TL2_SOCH2.
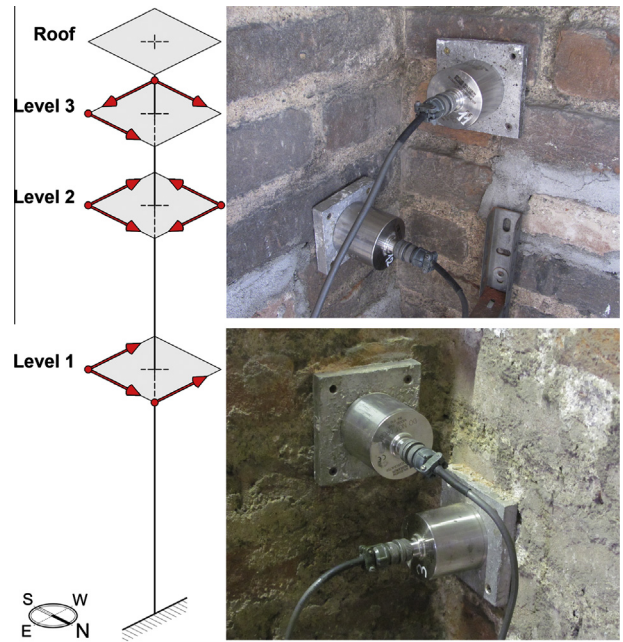

Fig. 13. Accelerometers layout during the dynamic tests and typical mounting of the sensors.
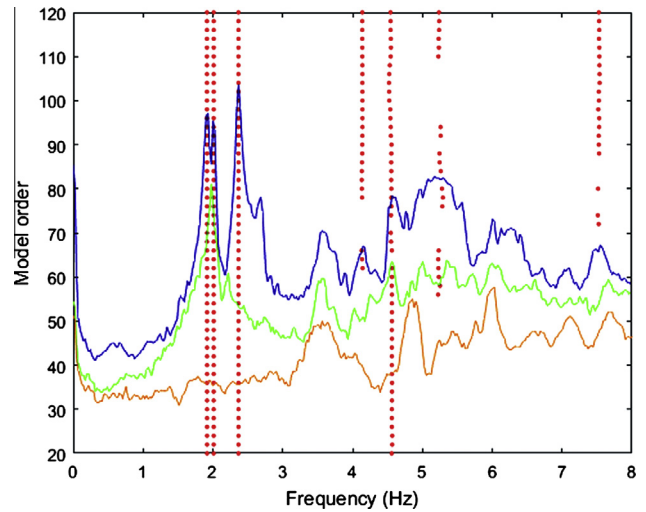

Fig. 14. Stabilization diagram obtained by applying the SSI-Cov technique to 1-h dataset and identification of natural frequencies.

modes since the deflections of the fourth mode (Fig. 15d) are negligible at this level – is the higher one suitable to the installation of a continuous dynamic monitoring system.

The second series of AVTs was performed between October 2nd and October 9th, 2015 and 166 datasets of 3600 s were recorded at a sampling rate of 200 Hz. As the accelerations induced by the swinging of bells do not comply with some basic assumptions of output-only modal identification, each dataset was firstly pre-processed with the aim of recognizing and extracting the time series associated to the swinging of bells. Hence, one "bell-free" time window of 3000 s was extracted from each 1-h dataset and, after low pass filtering and decimation, the SSI-Cov technique [28] was applied to identify the modal parameters.
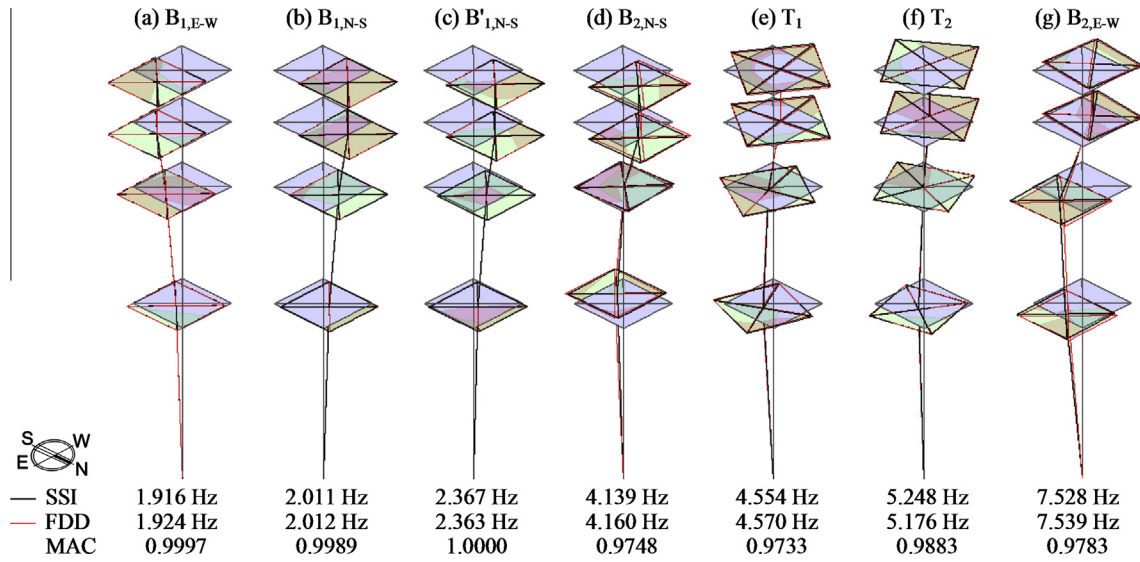
Fig. 15. Vibration modes identified from ambient vibration data.

	(a) $B_{1,E-W}$	(b) $B_{1,N-S}$	(c) $B'_{1,N-S}$	(d) $B_{2,N-S}$	(e) T_1	(f) T_2	(g) $B_{2,E-W}$
— SSI	1.916 Hz	2.011 Hz	2.367 Hz	4.139 Hz	4.554 Hz	5.248 Hz	7.528 Hz
— FDD	1.924 Hz	2.012 Hz	2.363 Hz	4.160 Hz	4.570 Hz	5.176 Hz	7.539 Hz
MAC	0.9997	0.9989	1.0000	0.9748	0.9733	0.9883	0.9783

The statistics of the natural frequencies, that were identified between 02/10/2015 and 9/10/2015, are summarized in Table 1 through the mean value (f_{ave}), the standard deviation (σ_f), and the extreme values (f_{min}, f_{max}) of each natural frequency; furthermore, the identification rate is reported in Table 1 as well. It should be noticed that: (a) notwithstanding the low level of the ambient excitation that existed during the tests, the lower 3 modes were identified with high occurrence, with the identification rate ranging between 97.0% and 99.4%; (b) the natural frequencies of the lower modes exhibit slight variations, with the standard deviation varying between 0.013 Hz (mode $B_{1,E-W}$) and 0.006 Hz (modes $B_{1,N-S}$ and $B'_{1,N-S}$); (c) the identification rate of the upper (torsion) modes turned out to be larger than 80%, even if the corresponding standard deviations ranges between 0.034 and 0.054 Hz.

Fig. 16a shows the time evolution of the natural frequency of the first bending mode in the E-W direction (mode $B_{1,E-W}$, Fig. 15a) along with the outdoor temperature: as it had to be expected from previous studies [18,24] on masonry towers, the modal frequency clearly increases with increased temperature, as it is further demonstrated in Fig. 17a where the frequency is plotted versus the (hourly averaged) outdoor temperature. It is worth recalling that the direct correlation between modal frequencies and temperature can be explained through the closure of minor masonry discontinuities, superficial cracks (Fig. 11a and b) and mortar gaps induced by the thermal expansion of the materials.

The frequency-temperature correlation is completely different for the first bending mode in the N-S direction (mode $B_{1,N-S}$, Fig. 15b), as shown in Figs. 16b and 17b: it can be observed (Fig. 16b) that the peaks in the modal frequency correspond to the valleys (relative minima) of the temperature. Furthermore,

the temperature-frequency plot reveals that the best fit line tends to be non-linear (Fig. 17b). In the authors' opinion, this unusual behaviour again might be related to the construction phases and confirms concern on the connection between the different parts of the building.

In order to explain the behaviour exhibited by the frequency of mode $B_{1,N-S}$, it should be reminded of the metallic tie-rod, which is placed between Level 2 and Level 3 to improve the connection between the North and South load-bearing walls (Fig. 12a); hence, the tension force in the tie rod, increasing with decreased temperature, might conceivably be a dominant driver of the changes observed in Figs. 16b and 17b. A possible explanation of the observed trend follows: (a) when the outer temperature is in the range 10–18 °C, the natural frequency tends to increase with increased tension force in the tie-rod (induced by the decreased temperature); (b) as the temperature increases, the loss of tension in the tie-rod is balanced by the closing of the micro-cracks and the "compacting" of the materials and those latter effects tend to become dominant for T > 23–25 °C.

As shown in Fig. 16c, the estimates of the natural frequency of the third mode (mode $B'_{1,N-S}$, Fig. 15c) are characterized by variations that are rather limited and similar to those exhibited by the second mode (mode $B_{1,N-S}$, Fig. 15b); on the other hand, the temperature effects (Fig. 17c) were not as clear as the ones observed for the lower two modes.

As the second series of AVTs allowed to identify with high occurrence the lower 3 modes of the bell-tower (exhibiting interesting behaviour for the health monitoring) and also considering the possibility of increasing the knowledge of the historic building through the combined use of static and dynamic monitoring, it was decided to permanently instrument the tower with the same sensor layout (4 accelerometers at Level 2) adopted in the second series of dynamic tests. The continuous dynamic monitoring system is now active since late October 2015.

6. Conclusions and future developments

The research has shown the importance of a global approach in the assessment of historic buildings. The available information on historic evolution, masonry inspection and on-site survey could help in the damage interpretation but also to give information for further investigation. The research plan consists of 5 successive
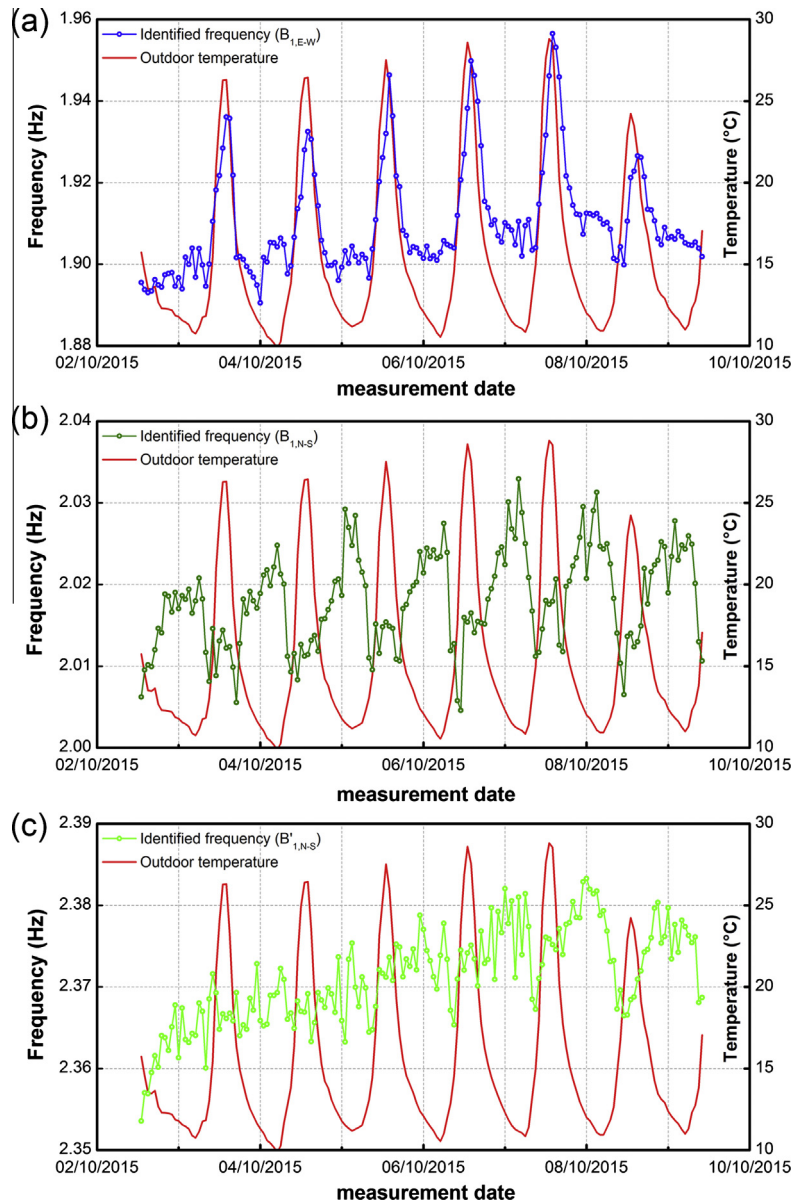
Table 1
Statistics of the natural frequencies identified (SSI-Cov) from 02/10/2015 to 09/10/2015.

Mode	f_{ave} (Hz)	σ_f (Hz)	f_{min} (Hz)	f_{max} (Hz)	Id. rate (%)
1 ($B_{1,E-W}$)	1.910	0.013	1.891	1.957	98.8
2 ($B_{1,N-S}$)	2.018	0.006	2.005	2.033	97.0
3 ($B'_{1,N-S}$)	2.371	0.006	2.354	2.383	99.4
5 (T_1)	4.549	0.034	4.493	4.681	87.4
6 (T_2)	5.139	0.054	5.024	5.295	80.1

B = bending mode; T = torsion mode.

Fig. 16. Variation of outdoor temperature and identified natural frequency between 02/10/2015 and 09/10/2015: (a) mode $B_{1,E-W}$; (b) mode $B_{1,N-S}$; (c) mode $B'_{1,N-S}$.

phases: (1) prompt on-site investigation, aimed at defining the main structural problems; (2) static monitoring of the main cracks; (3) operational modal testing aimed at identifying the dynamic characteristics of the tower and highlighting the most meaningful positions to be permanently instrumented; (4) development of a FE model of the building and vibration-based structural identification of the uncertain parameters of the model; (5) design and installation in the tower of a continuous dynamic monitoring system.

The first 3 phases have been described in the paper, whereas the above tasks 4–5 are still ongoing. Based on the results of prompt pre-diagnostic survey, static monitoring and extensive ambient vibration tests, the following conclusions can be drawn:

(1) In the case of the bell-tower of the Church *Santa Maria del Carrobiolo*, accurate on site survey of the building allowed to recognize the weakness of the structural layout, with two sides of the tower being directly supported by the load-bearing walls of the church apse and South aisle, and

suggested the subsequent static monitoring and dynamic testing.

(2) The static monitoring system, installed in the tower to survey the main cracks and structural discontinuities due to the building evolution, at present has not identified any abnormal increase of the crack openings, that have been so far dominated by the thermal effects.

(3) The static monitoring revealed that one main crack tends to open with increased temperature, as a consequence of the effect exerted by the metallic tie-rod installed above the crack and connecting the North and South load-bearing walls. The opening of the crack with increased temperature is associated with the relevant loss of tension in the tie-rod.

(4) The ambient vibration tests, performed by using an innovative radar vibrometer, provided the evidence of a slight decrease (from about 1.93 Hz to 1.88 Hz) of the fundamental frequency when the structure is excited by "random" swinging of the bells.
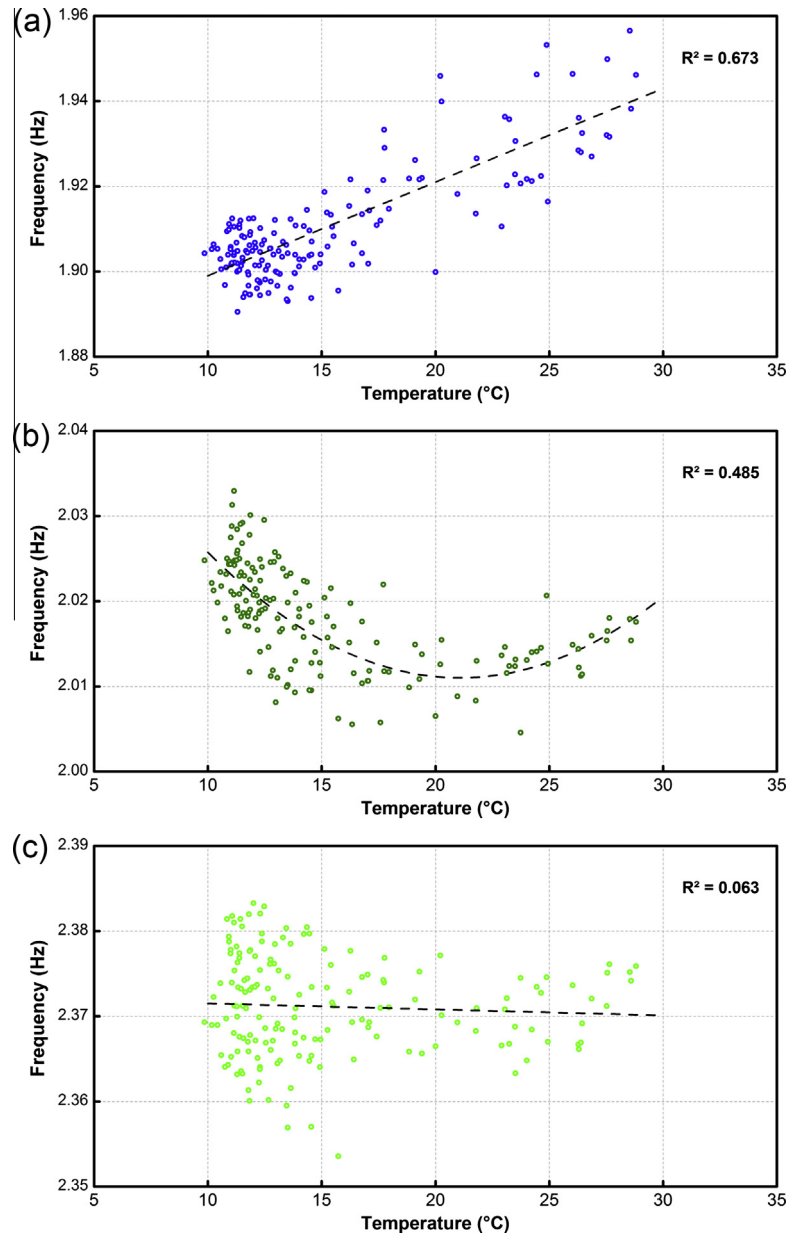
Fig. 17. Variation of identified natural frequency vs. outdoor temperature between 02/10/2015 and 09/10/2015: (a) mode $B_{1,E-W}$; (b) mode $B_{1,N-S}$; (c) mode $B'_{1,N-S}$.

(5) Notwithstanding the very low level of ambient excitation that existed during the dynamic tests performed by using conventional accelerometers, 7 vibration modes of the bell-tower were successfully identified within the frequency range 0–8 Hz.

(6) The dynamic characteristics of the bell-tower, identified by applying the SSI-Cov technique to recorded data, turned out to be very different from those obtained in past experimental studies of similar structures and is conceivably related to the weak structural arrangement of the building. In more details, the tower exhibits two bending modes (both in the N-S direction), that are characterized by closely spaced frequencies and very similar mode shapes, and two torsion modes, again with very similar mode shapes.

(7) The continuous monitoring of the dynamic response for 166 h allowed to verify that the temperature is a dominant driver of daily fluctuation of the natural frequencies and pro-

vided clear evidence of a rather complex and unusual dependence of the natural frequency of the second mode on temperature.

It is further noticed that:

(a) The information collected in the dynamic tests will provide a sound basis for the development and validation of a FE model of the tower. A rational methodology for the identification of the uncertain structural parameters of the model – previously developed by the Authors [17] and combining systematic manual tuning, sensitivity analysis and simple system identification algorithms – will be adopted.

(b) The investigation plan is completed by the installation of a continuous dynamic monitoring system (active since late October 2015), as a conclusive part of the health monitoring process helping the preservation of the historic structure.

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