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Vibration-based monitoring and diagnosis of cultural heritage: a methodological discussion in three examples

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ABSTRACT

Modern monitoring techniques contribute to accurately describing the structural health conditions of historical buildings and to optimising the plan of maintenance as well as the restoring intervention. Particularly, dynamic testing gives knowledge about global structural behaviour and can be used to calibrate numerical models and to predict the response to dynamic and earthquake loading. In some circumstances, vibration-based monitoring can also help in evaluating safety conditions.

The present paper proposes a discussion about the methodological multidisciplinary approach to modal testing when applied to architectural heritage buildings and structures, along with the description of selected case studies. These examples were chosen to cover the various issues connected to test design and interpretation.

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KEYWORDS: (architectural heritage, modal testing, experimental modal analysis, seismic response, dynamic characterization, geometric survey, model updating)

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1. INTRODUCTION

The World's architectural wealth includes a large amount of historical and cultural heritage buildings that are located in seismic risk areas. The structural monitoring and diagnosis of these structures is as important as it is necessary, owing to their intrinsic vulnerability, but also in view of preservation needs, which must be tempered with structural safety requirements and human safeguard.

The international deontological guidelines (ICOMOS, 2003) define the process of the structural rehabilitation of heritage structures similarly to the treatment of human diseases: "the heritage structures require anamnesis, diagnosis, therapy and controls, corresponding respectively to the search for significant data and information, identification of the causes of damage and decay, choice of the remedial measures and control of the efficiency of the interventions", an operation that certainly requires a multidisciplinary approach.

On the basis of these principles, the Italian seismic standards, based on the "Guidelines on the assessment and mitigation of seismic risk of the cultural heritage" (Moro, 2007), suggest an appropriate “level of knowledge” as being a key factor for its conservation, which can only be obtained through extensive investigations. More specifically, a historical building requires a knowledge path divided into several steps: the general identification of the structure and its environment factors, the collection of geometric and structural data, the identification of the materials and the survey of their state of conservation, the historical documentation, the mechanical characterization of the materials by means of various investigation techniques, the soil and foundation analysis and the relevant monitoring. As a matter of fact, dealing with ancient structures requires special attention during the documentation process, that has to be as thorough as possible, mainly because of factors such as the intrinsic characteristics of the masonry, the construction defects, the irregularities, the deterioration or failures caused by external forces: such properties make each of these buildings unique and lead to a higher degree of complexity when interpreting the structural behaviour.

The same standards point to the importance of periodic controls of the construction as the primary tool for preservation. Structural monitoring allows the planning of maintenance operations and the timely enacting of repair interventions in the case of structural damage, and consolidation for the sake of prevention only when it is really necessary (Moro, 2007).

A structural health monitoring (SHM) system is the result of the integration of several sensors, devices and auxiliary tools, such as: a measurement system; an acquisition system; a data processing system; a communication/warning system; an identification/modelling system; a decision making system.

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Even if it is based on innovative measuring, analysing, modelling and communication techniques, SHM shares the same goals as traditional monitoring methods. In fact, diagnostic monitoring can be considered as an extension of the well-established investigation practices since it integrates these novel technologies into a unique smart system. SHM tries to overcome the limitations of traditional visual inspections.

The traditional survey methods are affected by a large series of technical drawbacks. Visual inspections are generally not performed frequently enough, which risks affecting their predictive nature. Moreover, they are neither exhaustive, because they do not allow the hidden defects or the invisible effects of an on-going damage process to be detected, nor are they objective, because the estimation is related to the subjective judgement of an expert who can be fallible. More specific and accurate non-destructive testing (NDT) techniques are carried out off-line and usually only after the damage has been located (Shull, 2002). This means that, in the meanwhile, an excessive level of deterioration could have been reached. Moreover, non-destructive evaluations are often performed in a local manner and so provide information that only refers to a limited portion of the structure.

Modern diagnostic monitoring systems are created with the prerogative of overcoming these limitations by providing an exhaustive depiction of the structural health state and easing the plan of maintenance and restoring interventions. Among NDT techniques which can be easily implemented for online monitoring, we can start by mentioning the geometric control. In fact, techniques such as laser scanner or photogrammetry easily allow for online monitoring of the

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geometry of the building. Local control can be performed on particularly sensitive points (such as strain gauges for crack openings or wall tilting) with ad hoc sensors.

In the same class of NDT techniques, acoustic emission monitoring is arguably based on the simplest physical concepts, but is one of the most difficult techniques to implement in a practical way. A formal definition of acoustic emission is often given as ‘‘the release of transient elastic waves produced by a rapid redistribution of stress in a material.” The application of acoustic emission to non-destructive testing of materials typically takes place between 100 kHz and 1 MHz, using tools specifically designed to work in the ultrasonic regime.

Vibration-based SHM techniques, which constitute the object of this paper, have been successfully used for damage identification in existing structures. However, many issues require further investigation and still represent challenges that have to be undertaken. A new philosophy must therefore be pursued, which comprises the importance of a rational design of the monitoring system. It must integrate a sensors network which is capable of carrying out a continuous or periodic surveillance and providing reliable analyses based on different information sources. The environmental and operating condition variability must also be taken into account. In the past, relevant examples of dynamic monitoring included, among others, bell-towers (e.g. Bonato et al. (2000), Ivorra and Pallarés (2006), Casciati and Al-Saleh (2010)), arched structures (e.g. Ruocci et al. (2009), Ramos et al. (2010)) or domes and other monumental structures (e.g. Pau and Vestroni (2008), Chiorino et al. (2011), Boscato et al. (2012) and Russo (2012)).

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Whatever the monitoring approach and technique, when designing a SHM system, it is necessary to first perform an accurate analysis of the structural behaviour, in order to monitor the most expressive and sensitive parameters.

The intent of the present paper is a methodological discussion by way of a number of recent experiences on modal testing that shared the same approach, and whose common matrices are: the process of documentation, the numerical modelling, the test design, the testing campaign (acceleration measurements under ambient excitation), and the model updating. In spite of being heterogeneous from both the morphological and the typological point of view, three buildings were subjected to a typical model-driven vibration-based SHM, which included extensive surveying operations, ambient vibration dynamic investigations and numerical model calibration and verification.

2. VIBRATION-BASED STRUCTURAL HEALTH MONITORING FOR CULTURAL HERITAGE

Dynamic tests have been confirmed as an efficient investigative tool: they allow us to go back to the structural behaviour of the building system with reduced costs and minimally invasive actions, which is important when the necessity to preserve the material integrity becomes truly significant; moreover, in comparison with other investigation techniques, they provide information on the global behaviour of a structure. According to Farrar and Worden (2007) vibration-based SHM approaches may be classified in to two main groups, the data-driven and the model-driven.

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Data-driven approaches (Figure 1) are usually applied to data coming from permanent monitoring systems. In this case, a statistical model of the system is easily defined, and noise levels and environmental variations are established naturally. Conversely, approaches that are driven by high-fidelity physical models of the structure can potentially work without a validated damage model, but noise and environmental effects are difficult to incorporate.

System identification aims at extracting information about the structure's attitude to respond to dynamic loading, and it is the core of any model-driven SHM approach (Figure 2). In permanent monitoring systems, varied or anomalous parameters are directly associated to damage, and reliability can be defined as a function of identified quantities that reflect the damage, referred to as symptoms. Alternatively, a numerical model (e.g. a FE model) can be updated on the grounds of the identified parameters using model updating techniques which can be either direct (single step correction, e.g. Lagrange multipliers) or indirect (recursive minimization of a penalty function). Since in typical structural problems safety assessment relies on mechanical models, the engineer is prone to basing any evaluation, prognosis or decision on results coming from an updated model, rather than on symptoms. Accordingly, tests on the structures in examples should be supported by numerical models: the information gathered during the historical information phase and the survey documentation is translated into a FE model.

In actual practice, the uncertainties of masonry structures translate into great difficulties in defining a modelling method for a generalised application: although a mechanical model could be an important support to the dynamical test design, the deviation of the preliminary model

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results from the real behaviour can be significant. For this reason the results of the numerical analysis have to be reconciled with the experimental data carried out from dynamical tests, by means of consolidated model-updating techniques. Vice versa, the updated model can clarify, downstream the optimisation, some less certain aspects of the experimental data. The historical information and the survey documentation are themselves a support to the creation of a finite element model, as well as a comparison term with the model-updating results.

3. HISTORICAL INFORMATION AND BUILDING SURVEY

Architectural heritage structures require an accurate process of documentation to gather any useful information, as well as a multidisciplinary approach to clarify the most controversial aspects.

The historical documentation process consists of collecting all significant data coming from the available sources, such as registries and archives: original drawings, the description of relevant historical events and intervention evidence is all collected and compared. Not only does the knowledge of a building's history provide for a correct delineation of the stylistic features and its architectural character, but it also permits the identification of construction phases and possible interventions which, from the structural point of view, are important elements that may not necessarily emerge clearly from surveys and experimental tests.

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A detailed survey provides, on the other hand, the geometric and dimensional characteristics of the structure, as well as the classification of the elements, the materials employed and their state of preservation. The information gathered constitutes the basis for the creation of the simplified geometric representation to be used for the construction of the mechanical finite element model.

The three cases under consideration (see Figure 3) exhibit marked differences for what concerns typology, dimension, morphology and artistic relevance. As a matter of fact, during the process of historical documentation and geometric characterization they showed different levels of complexity. The three architectures are hereinafter described from the historical and morphological-dimensional point of view, on the basis of the information gathered during their documentation processes.

The Church of “Santa Maria delle Grazie” in Casale Monferrato, better known by its earlier designation of “Santa Caterina”, with its elegant baroque style, can be considered one of the most important religious architecture of that territory. The history of the Church begins with a donation by the marchioness Anna d’Alençon to the Dominican Sister of “Santa Caterina da Siena” in 1528: the religious order relocated to a new palace and built an annex which corresponds to the current day choir. In the Eighteenth century the church was renewed by Giovanni Battista Scapitta: he designed the extension as an element placed side by side to the old oratory (the choir body), and the building was completed only after his death, presumably by Giacomo Zanetti. The morphology of the church (Figure 4) is certainly the most complex among the three experiences (Figure 3), because of the dimensions of the centrally planned main body and the refinement of the baroque style composition. It is mainly characterised by a large oval dome (10 x 15 x 4.5 meters) supported by a 7 meters high drum. The building includes also a small atrium and an apse that communicates with the annexed choir body. The dome, supported by 8 masonry arches and by as many buttresses, presents a frescoed interior with an evident deterioration process. Over the dome, a slender 5 meters high lantern stands on 8 masonry pillars, one for each buttress, recalling the main structure, and it is finally covered by another little masonry dome. The facade, one of the most richly decorated elements, juts out from the church body of about 6 meters. The old choir building (10 x 22 meters) is covered by a barrel vault, supported by regularly spaced round arches in correspondence to the columns and reinforced with metal ties. The whole structure is realised in masonry. Only visual inspections were allowed on this building: thanks to a special spider platform also inaccessible parts were accurately examined. The survey confirmed that material degradation and cracks were limited to few portions of the building.

The Cathedral of S. Giovenale in Fossano (Cuneo province), a neoclassical building designed by Quarini in 1771, has a bell tower in a different style, because it is the only survived part of the earlier church, built in the Thirteenth century. In this case the geometry (Figure 5) is simpler than the previous experience, and the main complexity lies in its large size. The belfry, with a square base up to 35 m in height, is composed by masonry walls of the average thickness of 1.5 meters that contained the old passages for the stairs collocation, partially closed during the early Twentieth century, replacing the vertical connection structure with other internal ones made of wood. Three ceilings are interposed to the stairs system: the first one at a height of 9.9 meters, made of masonry, is supported by the Thirteenth century groined vault; the second one, built of wooden elements, is at a height of 28.2 meters, while the last one was built 32 meters high, with

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a mixed masonry-wood stratification. The same intervention included also three order of reinforcing metal rods, located respectively 14, 21 and 32 meters high. Above the level of the "Serliana"-style window stands the bell cell, with its masonry walls that are about 0.5 meters thick. The roof reaches the maximum level of the structure, which is 46 meters high. From a superficial inspection the masonry walls of the bell tower appears to be regularly coursed and in fairly good condition, but a core drilling campaign (whose results are listed in Figure 16) has shown that the tower is partially filled by rubble masonry. In particular, the middle and the top of the tower were built with poor quality materials.

The little votive church of "Madonnina della Neve" in Savigliano (Cuneo province), is the simplest of the three reported structures from the dimensional point of view; on the other hand it presents a considerable degree of morphological complexity, because of the multi-level interaction among its different bodies: the main hall (8.4 x 11.8 meters, maximum height: 9.60 meters), the apse, the bell tower (about 1.5 x 1.5 m, from about 5.9 m to 13 m high) and the rectory (see Figure 6). This building has its origin in 1609, when the middle class citizen Tommaso Ghigo promoted the building by raising the necessary funds. In 1618 the municipality added the bell-tower to the church, by means of its particular wooden beam system support. There is no other relevant information about the structure until 1763, when the church was radically transformed into its current aspect. In 1841 the consequences of a flood required some other restoration interventions. In the following year the service bodies were annexed with the present configuration. The church is based on central-plan geometry (Figure 6), with a central oval dome (7.8 x 4.5 x about 0.6 meters) without the drum, supported by 4 arcs. The back-side apse connects the main hall with the rectory, from which it is possible to reach the base of the

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roof trough the bell tower that does not rest upon the ground. The neoclassical façade (12.5 meters high) is the most decorated element and juts out the ledge level (9.36 m). Visual inspections confirmed that the masonry of the church is interested by a generalized cracking, in particular a severe crack pattern has been found in the main oval vault and at the connection between the façade and the lateral walls.

4. DYNAMIC INVESTIGATION TESTS

The behaviour of the three structures was investigated through dynamic tests. In particular, the pursued model-driven approach may be described in the following phases: the realisation of both a reference geometrical and a mechanical model; designing the test; on-site testing; signal pre-processing; structural identification; model-based diagnosis/prognosis/decision. Although the approach is the same for each application, in practice the marked architectural differences among the cases required specific solutions, especially for the design of the test campaigns.

4.1 Preliminary model

Firstly, a simplified geometrical model was built on the basis of the information obtained from the survey activities. Geometric information must not be too detailed because of the computational intensiveness of mechanical models; therefore the unnecessary geometric particulars are not to be included, since they are unnecessary for the solution of the problem (Figure 7). Moreover, when the geometric data are assimilated by finite element (FE) mechanical models, the required homogenisation of material properties nullifies any possible improvement associated to the geometric accuracy. Conversely, other information from the survey may become extremely important, such as the contact/interaction between different blocks/bodies, the presence of cavities or dislocations, remote parts and components, etc. (Roca et al., 2010).

The geometrical modelling of the Church of "S. Caterina", the most articulated structure, was certainly burdensome. The church of "Madonnina della Neve" presented some difficulties too, mainly related to the particularity of the multi-level connections among the different bodies of the building aggregate. On the contrary, despite its dimension, the geometric representation of the bell tower of S. Giovenale was simpler than the other two ones, because of its less complex morphology.

These geometrical virtual simplifications were made using the 3d-modelling software RHINOCEROS, and subsequently they were employed as a first input in the realisation of linear-elastic models of the three structures, using the finite element package ANSYS. Figure 8 shows the three FE models: it is evident that only the most important elements were represented. For instance, each wooden roof was excluded from the model. Consequently, the roofs were considered as a concentrated or distributed mass depending on the case.

The mesh for FE modelling was performed using bi-dimensional elements (SHELL) for walls, vaults and domes. Mono-dimensional elements (BEAM) were employed to model pillars, columns, groins, ties, rods and beams. Punctual elements (MASS) were also inserted in some cases. Structural cracks were not considered in the FE model, as their effects on the order of the presentation of the natural modes of the structure were considered to be virtually negligible.

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The structures were assumed to be clamped at the base. Furthermore, in these linear FE models, the continuum underlying the structure was disregarded, based on the assumption that it has no important effects on vibration modes. In a more general case, soil-structure interaction should be carefully examined (e.g. resorting to a sensitivity analysis) and, where possible, included in the model.

The mechanical model of S. Caterina church contains about 18000 elements and 17000 nodes. The whole structure was modelled: the church, the choir and the annexed open gallery. The adjacent body was firstly considered a rigid constraint. The chosen material model was a linear elastic isotropic constitutive law, initially uniformed on certain parts of the structure, with a Young modulus of 2500 MPa, a Poisson's ratio of 0.4 and a density of 2000 kg/m^3.

The model of the Fossano bell tower has about 7500 elements and 15000 nodes. In the general case, the mechanical relationships between the tower and the church must be carefully evaluated and special elements or springs should be introduced and calibrated in order to simulate the interaction between the two bodies (De Stefano and Ceravolo, 2007). After an accurate survey, the connection walls with the church of S. Giovenale were modelled assuming a lack of efficacy of constraints. The elastic material model adopted at the beginning had a Young modulus of 2000 MPa, a Poisson's ratio of 0.4 and a density of 2200 kg/m^3. To simulate the mentioned weak connection state with the main structure, a fictitious material was assigned to that portion, which has Young's modulus of 500 MPa.

The model of the "Madonnina della Neve" church was realized with 11000 elements and ca. 11000 nodes. The complete building aggregate was modelled with the same criteria used for the

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other two examples; the masonry was assigned a Young modulus of 2000 MPa, a Poisson's ratio of 0.4 and a density of 1700 kg/m^3. The particular condition of the bell tower required the introduction of support wooden beam elements, with a Young modulus of 11000 MPa, a Poisson's ratio of 0.35 and a density of 600 kg/m^3, as well as some CONTACT elements to model its connection with the apse vault.

4.2 Dynamic tests design and testing campaigns

The dynamic tests were designed to maximise: a) the spatial resolution of experimental modal shapes associated with the main structural response; b) the number of modal parameters (modal frequencies, damping ratios) later used for calibrating the mechanical models. Moreover, experimental setups depend on the specific scope of the investigations. For instance, the setups designed to capture low frequency global modes of a building provide information on the overall condition of the building and they are particularly helpful in the prediction of the seismic response. Conversely, higher frequency modes are more sensitive to local condition changes or localized damage. Accordingly, the preliminary FE models were used to identify the structural portions and the elements that mainly affect the dynamic behaviour of the structure. Based on the FE vibration modes, the measuring points (henceforth "setups") were defined, as well as acquisition settings and parameters. The optimal location of sensors can be trivial in structures with simple geometry and the problem can be solved simply relying to the experience of the operator, or with a trial-and-error approach. However, architectural heritages are often complex structures calling for more sophisticated strategies, such as optimal sensor placement (OSP). One of the most widely used approaches has been proposed by Kammer (1991) and consists in a minimisation of the norm of the Fisher information matrix. Another way to solve the problem is the modal kinetic energy-based method as proposed by Salama (1987) and Chung (1993). A deep review of OSP methods with particular emphasis on vibration measurements can be found in Li *et al* (2007a, 2007b).

The test design had to also consider limitations both in the number of sensors and in the accessibility of the buildings. Therefore, a number of conjectural setups were generated and verified, aimed at maximising the decoupling of the modal shapes. This was done using simple Auto-MAC (Modal Assurance Criterion) indexes (Allemang, 2003) and rules. A robust approach in the solution to the OSP problem is the one proposed by Yi *et al* (2011) which relies on a QR factorization for the initial sensor placement, and then in a minimisation of the off-diagonal terms in the Auto-MAC matrix with a genetic algorithm.

Figure 9 exemplifies the design of the sensor setup for the church of “Madonnina della Neve”. In this case, the preliminary modal analysis allowed investigating the dynamic behaviour of the structure, so supporting the choice of the most sensitive locations where to place the 18 sensors. Without the indication of any FE modal analysis, it proves to be very difficult to plan an efficient sensor placement, especially in case of a rigid and compact structure like the above-mentioned example.

The sensors used for the dynamic tests were uniaxial PCB Piezoeletronics capacitive accelerometers (Model 3701G3FA3G), with a sensitivity of approximately 1V/g, a measuring range included between 0 and 3 g, a resolution of 30 μg and a mass of 17.5 grams. The sensors were connected by coaxial cables to a Difa-LMS SCADAS data logger that provided for the amplification of signals, which in turn were sent to the laptop hard-drive, where the acquisition software is installed. The signals acquired in the three examples are related to the response of the structure to ambient noise excitation, produced by external stochastic forces (e.g. vehicular traffic, wind, micro-tremors, etc.).

Sensors are often in insufficient number to test the whole structure in a single step because of the morphological complexity or the size: for example, both for the S. Caterina church and the Fossano bell-tower the testing campaigns required to design four setups. In these cases, one of the setup has to be always online, permitting to assemble the acquired data during the signal processing phase, while the other setups concern the different parts of the structure (such as the lantern and the facade of S. Caterina, as suggested by preliminary FE model studies). In order to merge the eigenmodes identified on different setups, a subset of "link" sensors (typically from 4 to 6 accelerometers per setup) has to be kept fixed during dynamic tests.

Often the considerable size of historical buildings requires also some specific practical solution to place the sensors during the testing campaign: for instance, in the case of S. Caterina two different spider platforms were employed to reach the less accessible points (Figure 10).

Generally a trial acquisition proves necessary to test that the equipment is fully functional. It is also appropriate to annotate the serial numbers and to choose the directions of the sensors in a methodical way. Moreover, it is recommended a careful preparation, numeration and connection of the cables; indeed, it was shown by the experience that often inaccuracies in the execution of tests may affect the quality of the results. These last operations prove to be useful also with the perspective that the experimental data want to be made usable by operators who have not directly

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participated in the experience. In periodical monitoring activities, all sensor locations need to be photographed and archived.

The choice of the sampling frequency must allow capturing the response of the main vibration modes of the structure. In the three mentioned cases, the acquisition of the signals was carried out using a sampling frequency of 400 Hz, which corresponds to a useful band which spans from 0 to 200 Hz (according to the Nyquist criterion), whilst the main modes were confined in the 20 Hz. Since the acquired signals in all cases were generated by ambient excitation, their length is of great interest because it is necessary to perform numerous segmentations in order to improve system identification robustness. A signal of 500000 samples (approximately 20 minutes with sampling frequency equal to 400 Hz) can be considered sufficient for structures which typically have a range of fundamental periods lower than 1 s.

Preferably, a first experimental modal analysis from acquired signals should be executed on site, this allowing for a further optimisation of the setups, as well as for checking the correct working of the measurement system.

4.3 Structural Identification

Interest in output-only identification techniques mostly arises from the difficulties encountered in producing, and measuring, proper excitations in large-sized structures. In recent years, time domain techniques have been pursued rather successfully, also thanks to the great spectral resolution offered in the analysis of complex systems, and thanks to their modal uncoupling capability (Shinozuka et al., 1982, Peeters and De Roeck, 1999, Loh et al., 2000, Reynders et al., 2008, Juang and Pappa, 1986).

An important family of time domain methods uses autoregressive time series models and exploits formulations developed in the field of system control. Another strategy consists of employing methods well-tested in the identification of structures on the basis of impulse response or free decay, and extending them to the analysis of response signals generated by excitations of a more general nature. A third option refers to the Ho-Kalman minimal realization algorithm. These approaches, or their combinations, include a sizable proportion of the methods actually employed in “output only” identification of civil structures subjected to natural excitation, and most of them are based on stationarity assumptions. In order to overcome limitations associated to stationarity assumptions, special techniques were also conceived for non-stationary excitation (Yuen et al., 2002, Ceravolo, 2004, Wu and Smyth, 2008). Inaccuracy in damping estimation provided by standard output-only techniques has recently been reported and quantified (Ceravolo and Abbiati, 2013).

One of the basic shortcomings of these methods is that they often produce spurious modes, whose true nature, however, can usually be identified by means of simple modal form correlation indicators (Ewins, 2000), or, as an alternative, with the aid of numerical models.

All the mentioned techniques are incorporated in the Structural Dynamic Identification Toolbox (SDIT) code, implemented in Matlab environment at the Dep. of Structural, Geotechnical and Building Engineering of Politecnico di Torino (Ceravolo and Abbiati, 2013). As for the three concerned structures, the dynamic identification, in terms of frequencies, damping ratios and

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modal shapes, has been carried out in the time domain. The time-domain family of stochastic subspace identification methods stems from Ho and Kalman's classical realisation theory that was extended to stochastic systems by Akaike and Aoki. Van Overschee and De Moor (1996) collected and systematically linked together contributions from different fields: system theory, statistics, optimisation theory and linear algebra. The essential quality of all the algorithms within the subspace family is their ability to work out the matrices describing a linear system starting from subspaces containing the projections of data matrices; in particular, these algorithms project the space of the matrix rows of future outputs into the space of the rows of past outputs.

The technique used in the reported applications is the third algorithm considered by the unifying theorem of Van Overschee and De Moor (1996). This method, which is often referred to as "Canonical Variate Analysis" (CVA), was originally developed by Larimore (1990).

The signals acquired are conditioned using filters, de-trending and sub-sampling. After a pre-analysis with Welch power spectral representations (Welch, 1967), several time-domain identification sessions were executed. Computational modes are systematically discarded by using the MAC (Allemang, 2003). In greater detail, all signals coming from different acquisitions were segmented and a large number of SSI identification sessions were performed. In order to eliminate spurious eigenmodes, a cleaning criterion was adopted. First, eigenmodes characterized by damping values greater than 0.20 were automatically discarded. Then, both the sets of identified frequencies $\mathbf{F} = f_1 \quad f_2 \quad \dots \quad f_n$ and mode shapes $\mathbf{V} = v_1 \quad v_2 \quad \dots \quad v_n$ resulting from each acquisition segment were used to compute the Boolean affinity matrix $\mathbf{A}$ defined as:

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$$\begin{cases} \mathbf{A}_{i,j} = 1, \text{ if } MAC\left(v_i, v_j\right) \cdot \exp\left[-\left(\dfrac{f_i - f_j}{\sigma}\right)^2\right] \geq 0.95 \\ \mathbf{A}_{i,j} = 0, \text{ if } MAC\left(v_i, v_j\right) \cdot \exp\left[-\left(\dfrac{f_i - f_j}{\sigma}\right)^2\right] < 0.95 \end{cases} \tag{1}$$

where a frequency threshold σ equal to 0.20 Hz was assumed. An index of recursion C_i relevant to each single *i-th* eigenmode has been defined as follows:

$$C_i = \sum_{j=1}^{n} \mathbf{A}_{i,j} \ . \tag{2}$$

Eigenmodes characterized by $C_i > 5$ were kept, whilst the others were discarded as spurious. The threshold value of the index of recursion C_i was proportional to both the segmentation number and to the system order range considered and, in addition, to the noise level of acquired signals.

Stabilization diagrams were used to identify frequencies in each segment (stabilization criterion: maximum frequency deviation: 2%), then additional tolerance criteria were used for MAC (5%) and for damping ($0 \leq \zeta \leq 10\%$). By executing a statistical recurrence of the system's natural frequencies identified by the SSI algorithm and by averaging the values, it proves possible to distinguish between the real modes of the structure and modes that appear occasionally, and might be possibly due to exogenous components (Carden and Brownjohn, 2008).

The next step was a new identification process assisted by the FEM models, whose elastic moduli were tentatively corrected so as to match at least the fundamental frequencies. In this second identification stage, in fact, mode attribution was based essentially on a comparison between the modal shapes extracted from the signals and those obtained from the FEM. By means of the finite element code it was also possible to work out a qualitative classification of the modes, which otherwise would have been virtually impossible when dealing with highly complex structures.

The main results of the three identification procedures are herein described and summarized in terms of frequencies and damping ratios in Table 1.

The identification performed on “S. Caterina”‘s signals detected six natural main modes (Figure 11): the first one is a transverse modal shape (at 3.03 Hz); the second mode shows a longitudinal bending displacement (3.33 Hz); the third one is characterized by another transverse bending movement (3.95 Hz, dome and façade in phase opposition), while the fourth one has a longitudinal bending displacement (4.40 Hz dome and façade in phase opposition). The fifth mode (5.11 Hz) presents another transversal modal shape and the sixth (5.39 Hz) is a typical drum-dome mode along with a strong torsion of the façade.

In the case of Fossano’s tower, the stochastic subspace procedure estimated a total of 11 natural modes (range 1.28-9.65 Hz), of which the first two ones (1.28 Hz and 1.34 Hz) are bending modal shapes in the two horizontal directions, while the third one shows torsional features (3.28 Hz). Within the above mentioned frequency range, a few extracted modes are characterised by mutual movements of the four walls, but, more importantly, wall local modes (Figure 13) were detected at low frequencies, a condition that is associated with poor connections between walls and bad overall condition of the masonry.

The experimental modal analysis of the “Madonnina della Neve” resulted in the identification of 12 vibration modes of the structure. The two main translational modes (at 3.81 and 5.14 Hz), the third (6.01 Hz) and the fourth (6.66 Hz) modal shapes are characterized by phase opposition between the facade and the bell tower.

5. MODEL UPDATING

Vibration-based model updating consists in tuning the preliminary FE model by using the dynamic characteristics determined through system identification techniques, with emphasis on modal frequencies and shapes. The approach is consolidated (Friswell and Mottershead, 1995) and applied to a wide range of civil structures including, for instance, the updating of a bridge made by Zivanovic et al (2007); or, in the field of architectural heritages, the example of the Colosseum (Pau and Vestroni, 2008); and again, with the Sanctuary of Vicoforte (Chiorino et al., 2011). FE model updating helps to understand the dynamic behaviour of the structure but might also help for its damage assessment (Brownjohn et al., 2001, Bakir et al., 2007).

The number and the type of parameters to be calibrated must have an influence on the dynamical behaviour but, at the same time, they should not to make the procedure of calculation burdensome which could lead to unreliable results (Friswell and Mottershead, 1995, Mottershead et al., 2011).

The selection of parameters may be based on a preliminary sensitivity analysis. The parameters must however have a clear physical meaning (e.g. elastic modulus, mass densities and constraints) and are then varied, by means of optimisation procedures, until they converge to values that match what was recorded by the experimental evidence. In order to take into account the spatial distribution of mass and stiffness, the FE model is usually subdivided in macro-elements, with different materials, as shown by the example in Figure 14.

In the three reported examples, the elastic moduli were initialized to uniform the values for each element. Then the model updating procedure was based on the synergistic use of ANSYS and MATLAB software, employing an algorithm that investigates the minimum of a cost function, by varying the parameters associated with the macro-elements within a predetermined range.

The choice of the most suitable optimisation algorithm is not always a trivial task. A distinction can be made mainly between local and global search methods. The first type of methods requires a starting point to initialise the minimisation procedure. This constraint can be an advantage if an approximate evaluation of the parameter value has been made (i.e. experimental campaign or a previous optimisation). Among the types of methods it is worth mentioning the widely used quasi-Newton methods (Nocedal and Wright, 2006) or derivative-free algorithms such as direct search methods (Lewis et al., 2000). On the other hand, one may have a great degree of uncertainty on the starting value of the parameters: in such a case, stochastic optimisation methods are probably more suitable. Genetic algorithm (Michell, 1996) is probably the best known of the stochastic methods. When dealing with these algorithms it is advisable to rerun the optimisation process a few times in order to be sure that the global minima of the function has been found.

The results coming from the optimisation procedure should be carefully checked and compared with literature and reference values, as success in the optimisation procedure does not directly

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imply that the model has a clear physical meaning and has been verified. In other words, model updating may lead to a compensation of model structure errors by model parameter errors. These models then give good agreement between the measured and computed data but will lead to erroneous predictions.

The choice of the cost function depends on the particular needs. Equation 3 was employed for the optimization procedure of the FE models in the three examples:

$$f = \sum_{i=1}^{m} \alpha \left(\frac{f_{Ai} - f_{Ei}}{f_{Ei}} \right)^2 + \beta \left(\frac{1 - \sqrt{MAC_i}}{MAC_i} \right)^2 \tag{3}$$

where f_{Ai} and f_{Ei} represent the i-th numerical and experimental mode frequencies, respectively, whilst α and β are two weighting coefficients for frequency and modal shapes, respectively, and MAC_i is the modal assurance criterion for the i-th mode.

The cost function can be divided into two parts: the first one is a weighted frequencies delta while the second part is a weighted modal shape delta. The latter part allows the correct order of modal frequencies to be maintained. The choice of the weight ratio values depends on the emphasis assigned to different parameters, or simply on their accuracy. The values of the weighting coefficients can be typically assumed to vary between 0 and 1, and their final values can be determined with trial-and-error procedures (Merce et al., 2007).

Once the error function has been defined, it may be useful to perform a sensitivity analysis on the parameters affecting equation (3) and the model output (i.e. frequencies). Several methods are available to carry out a sensitivity analysis: from classic local methods based on derivatives

(Saltelli et al., 2000) to more general global methods (Saltelli et al., 2008). Using the results of the sensitivity analysis one can limit the parameters to be optimised to an acceptable number.

Constraints on the parameters should be added to the optimisation problem in order to limit the search space of the function. For instance, implausible values of the masonry elastic moduli have been excluded in the three examples (i.e. 1e8 Pa$<E_i<$1e10 Pa).

Downstream the described optimization process, generally constituted by several model-updating steps, the output data consist of a modified set of parameters related to the macro-elements of the different buildings, which reconciles the FE model to the experimental evidence, as exemplified in Figure 14 and Figure 15, in the three case studies. In Table 2 in particular, the comparison between the experimental modes and those provided by the updated FE model is expressed in terms of MAC matrix (for modal shapes) and per cent errors (for modal frequencies).

6. PREDICTION AND DIAGNOSTIC CAPABILITIES OF THE UPDATED MODELS

Updated models, as illustrated in the previous section, allow a reliable estimation of the mechanical properties of the materials. The elastic parameters, as related to the different macro-elements of the structure, provide direct evidence of anomalies affecting the structural behaviour, such as local weakening, cracking, dislocations etc., so helping a diagnosis and possibly a prognosis (e.g. in terms of residual life without interventions).

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Although updated models may indeed give a valuable indication of possible structural flaws to be investigated through local NDT techniques, one should be careful with directly applying these models for structural integrity assessment. In fact, the modal characteristics are often insensitive to local weak spots that are important for structural integrity. As a general rule, possible damage or weakening should be properly modelled and confirmed by physical evidence.

To this end, the case of "S. Caterina" is interesting, firstly because the distribution of the Young modulus indicates a substantial disconnection between the facade element and the rest of the church; secondly, an excessive deformability of the lantern, as indirectly revealed in terms of the reduced value of the equivalent elastic modulus, was then confirmed by the presence of severe cracking in the columns. On the contrary, the high values of the elastic modulus relative to the base of the church and the choir show the good condition of these elements, as confirmed by visual inspections.

The results of the Fossano bell-tower are even more interesting, since a comparison term was available, consisting of other experimental data gathered during a core-drilling campaign and visual inspection by means of an endoscope; as a matter of fact, the middle part of the tower has been recognized as the weakest element of the building, a feature confirmed by the analysis of core samples (Figure 17) and which is often found in masonry towers (Ferretti and Bazant, 2006). The deterioration of the masonry was also confirmed by an unfortunate event that happened a few days after the testing campaign: a heavy crack propagated in the lower walls. Concern for an incipient collapse convinced local authorities to declare the structure unfit for use.

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The results of “Madonnina della Neve” are also interesting, especially in comparison with those of “S Caterina”: the equivalent elastic modulus distribution, obtained downstream the model-updating procedure, highlighted a poor connection between the structural parts. Then, after a fine subdivision into macro-elements, the weaknesses were more precisely detected, as shown in Figure 17, and partially confirmed by visible cracks in the lateral masonry walls. The first frequency of the FE model moved from 6 Hz to 3.9 Hz: in fact the presence of cracks was not taken into account in the initial model. The global behaviour observed in this structure showed an unsuspected flexibility in both longitudinal and transversal directions.

On the basis of the cases studied, it is evident that dynamic testing on historical structures may lead to mechanical models with reliable prediction capabilities. In particular, updated models may be used for the purposes of structural health monitoring, safety assessment and prognosis. Despite their inherent linearity, updated models may still constitute a reference for predicting the seismic behaviour of monuments, by means of either time-history or response spectrum analysis (Figure 18), also in view of future interventions.

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7. CRITICAL ASSESSMENT OF VIBRATION-BASED MONITORING FOR CULTURAL HERITAGE

In the following, a critical assessment of vibration based monitoring of ancient heritage buildings is provided, in order to discuss and point out the drawbacks and limitations of the core methodology, and provide a few practical recommendations.

Experimental tests on historical buildings need to be carefully designed, based on the survey and the history of the structure under examination. The sampling rate and sensors depend on the specific scope of the investigations, whilst measurement setups can be optimized so as to capture low frequency global modes (e.g. seismic assessment), or possibly local dynamic features (e.g. detection of structural flaws to be followed up by the local NDT techniques). The excitation source is typically environmental noise (such as wind or traffic). This type of excitation cannot be measured, but presents several advantages with respect to forced excitation: there is no need to apply external loads or special devices and there are no risks of inducing additional damage to the structure. Moreover, environmental excitation virtually allows an unlimited set of data to be acquired because the input source is always present, this being an essential feature for permanent structural health monitoring systems.

A vast majority of the methods actually employed in "output-only" dynamic identification of structures subjected to environmental excitation work in the time domain and are based on stationarity assumptions. Whilst in frequency estimates all methods retain a satisfactory degree of accuracy, giving errors mostly below 1%, the assessment of damping is more critical, as the estimate of damping results as being unstable (with errors, in some cases, of over 100%). In actual practice, several sets of data need to be treated and statistically analysed. This notwithstanding, errors in damping estimation of current output-only techniques range between 15% (SSI) and 50% with virtually stationary response signals, and tend to increase with the level of non-stationarity (Ceravolo and Abbiati, 2013).

A numerical model is usually updated on the grounds of the identified parameters by using indirect procedures (typically recursive minimization of a penalty function). The parameters to be calibrated should be a limited number and should reflect the quantities with a clear physical significance (interaction with soil or other bodies, elastic moduli, masses, crack patterns, joints etc.). The results coming from the optimisation procedure should be compared with literature and reference values, because success in the optimisation procedure does not directly imply that the model is verified. Any possible damage or weakening should be properly simulated by the model and should then be confirmed by physical evidence or by more specific local investigations.

If all the assumptions and limitations are taken into account, vibration-based monitoring can contribute to the identification of the parameters that influence the structural response of the asset, can support a further stage of local investigations, and can significantly reduce the number of destructive tests, also in the light of modern approaches to the structural assessment of architectural heritage (Lagomarsino et al., 2012).

8. CONCLUSION

This paper has presented a methodological approach, which has been successfully pursued in the application of experimental modal analysis to cultural heritage structures via three very different case studies. The different phases involved in vibration-based monitoring, model updating and results interpretation have been reviewed, highlighting the peculiarities and difficulties that one may encounter when dealing with architectural heritage structures. The quality of the results achieved has been confirmed by extensive traditional tests, especially in the case of the Fossano bell-tower. Finally, the paper stresses the importance of having an updated FE model, even if linear, which can help the structural engineer in taking difficult decisions regarding the conservation of the structure.

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Figure 1. Structural Health Monitoring: Data Driven Approach

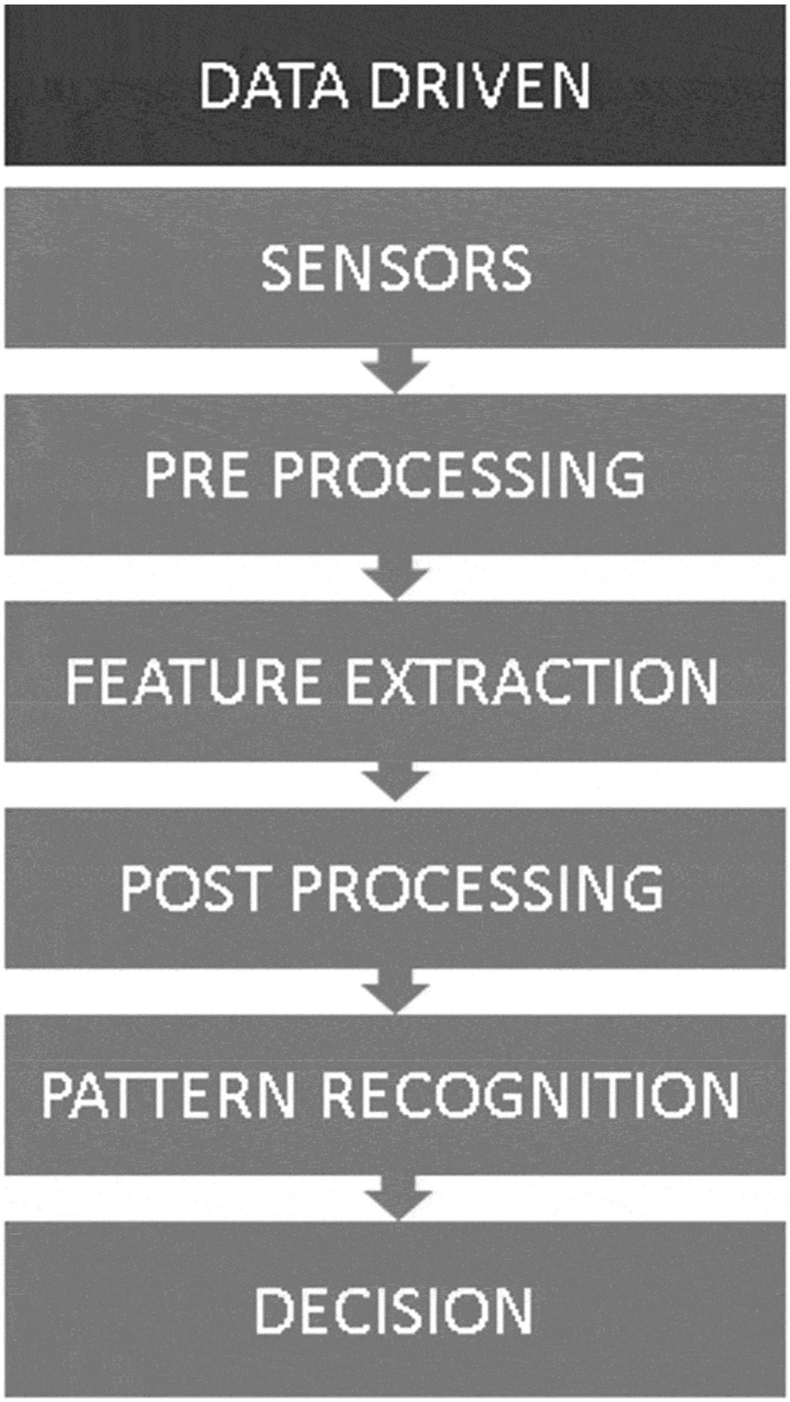

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Figure 2. Structural Health Monitoring: Model Driven Approach

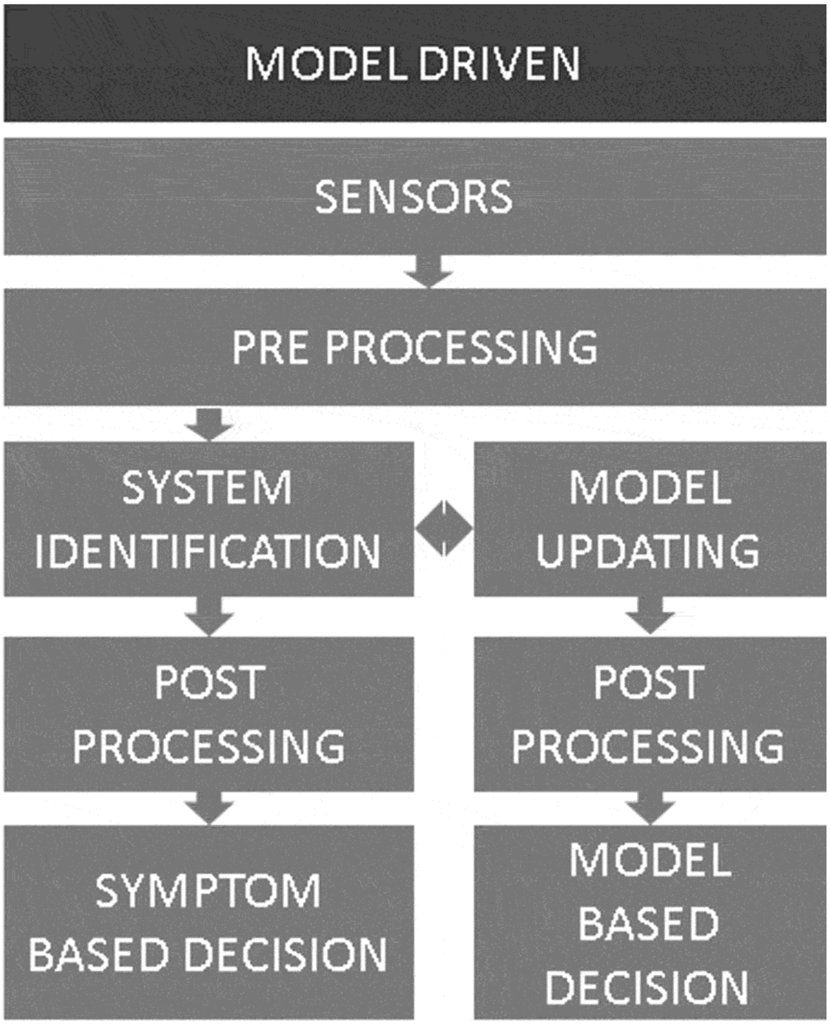

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Figure 3. Picture of the three reported architecture, from left to right: the church of "S. Caterina", the Fossano's Bell Tower and the church of "Madonnina della Neve".

Figure 4. Survey of the church of “S. Caterina”

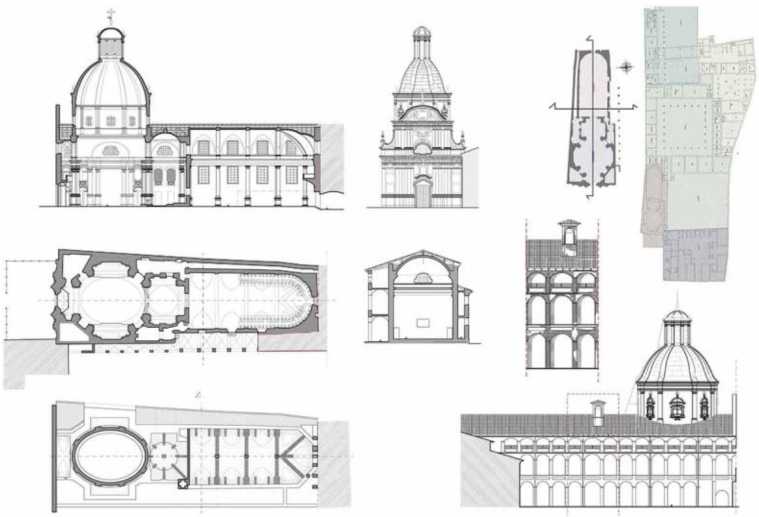

Figure 5. Survey of the Bell tower of "S. Giovenale"

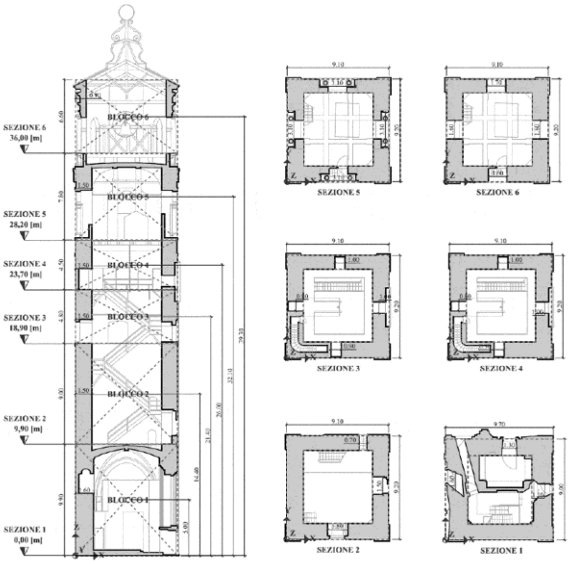

Figure 6. Survey of the church of "Madonnina della Neve".

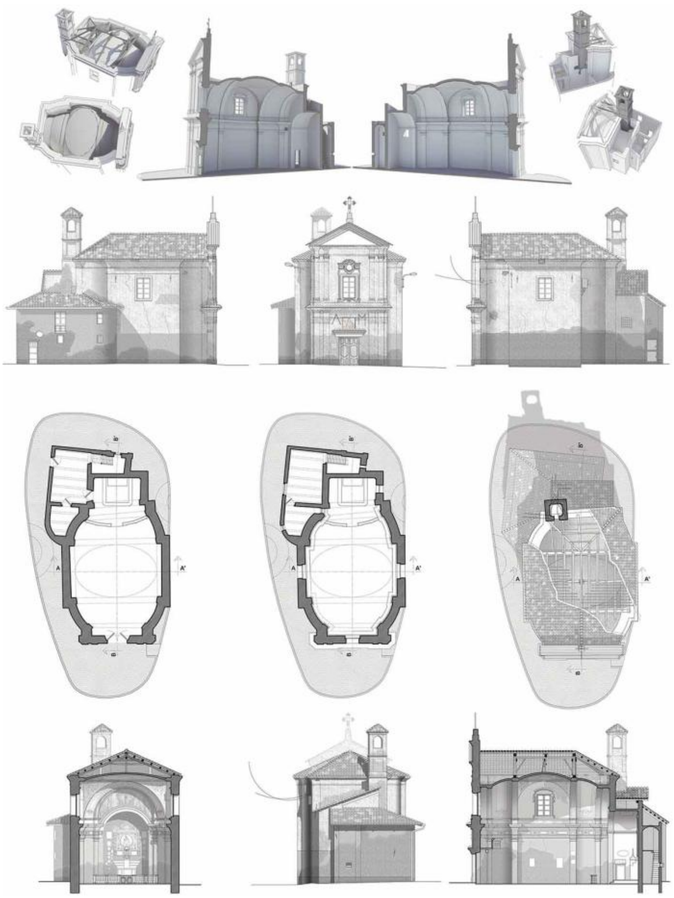

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Figure 7. The process of geometrical simplification: from 3d model to FE model in the case of the church "Madonnina della Neve"

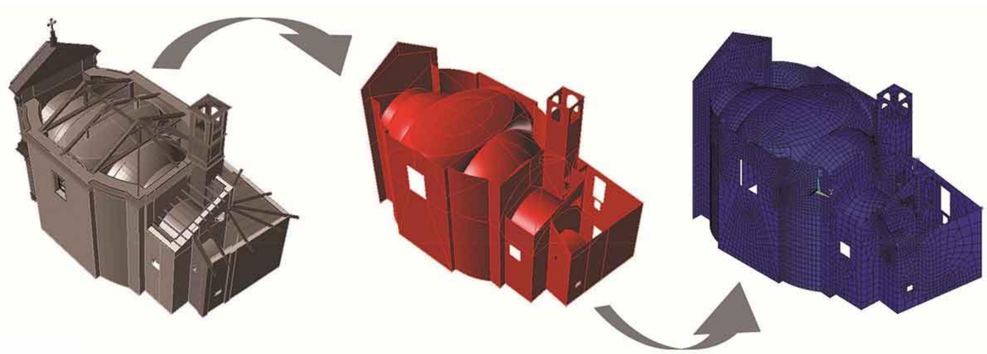

Figure 8. The three FE models, from left to right: “S. Caterina”, Fossano’s Bell Tower and the “Madonnina della Neve”

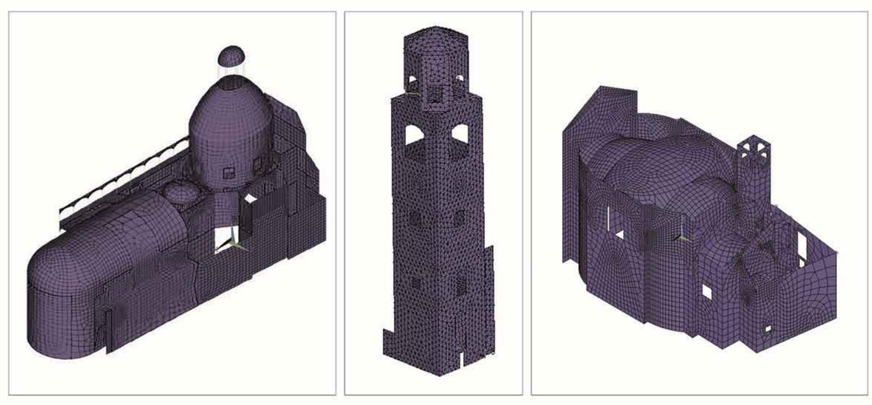

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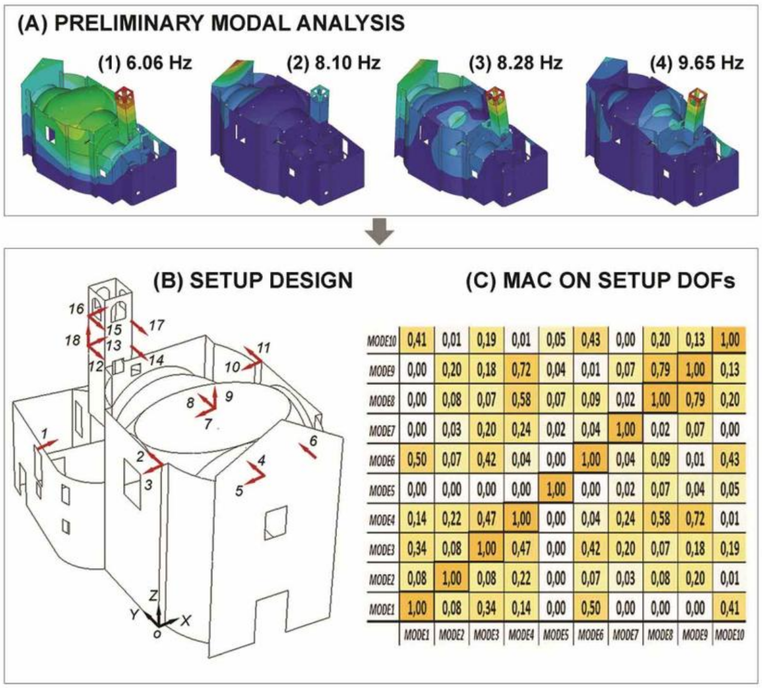
Figure 9. Dynamic tests setup: (a) preliminar modal analysis using a not calibrated FE model, (b) accelerometric setup design, using 18 channels, (c) MAC between modes in order to assess the decoupling capabilities of the setup.



	MODE1	MODE2	MODE3	MODE4	MODE5	MODE6	MODE7	MODE8	MODE9	MODE10
MODE10	0,41	0,01	0,19	0,01	0,05	0,43	0,00	0,20	0,13	1,00
MODE9	0,00	0,20	0,18	0,72	0,04	0,01	0,07	0,79	1,00	0,13
MODE8	0,00	0,08	0,07	0,58	0,07	0,09	0,02	1,00	0,79	0,20
MODE7	0,00	0,03	0,20	0,24	0,02	0,04	1,00	0,02	0,07	0,00
MODE6	0,50	0,07	0,42	0,04	0,00	1,00	0,04	0,09	0,01	0,43
MODE5	0,00	0,00	0,00	0,00	1,00	0,00	0,02	0,07	0,04	0,05
MODE4	0,14	0,22	0,47	1,00	0,00	0,04	0,24	0,58	0,72	0,01
MODE3	0,34	0,08	1,00	0,47	0,00	0,42	0,20	0,07	0,18	0,19
MODE2	0,08	1,00	0,08	0,22	0,00	0,07	0,03	0,08	0,20	0,01
MODE1	1,00	0,08	0,34	0,14	0,00	0,50	0,00	0,00	0,00	0,41

Figure 10. Different phases of the testing campaign of "S. Caterina": (a) choice of the excitation source, (b)-(e) sensors dislocation – a difficult task in some cases, (c)-(d) acquisition of the signals.

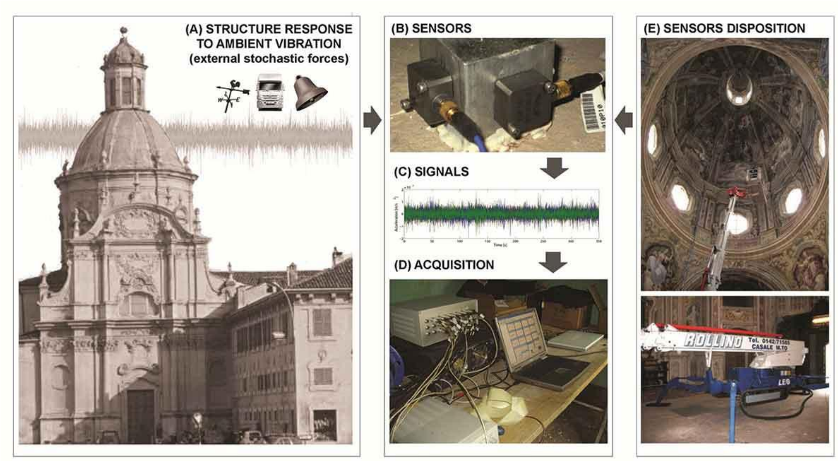


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Figure 11. (a)-(b) The software interface used for dynamic identification,

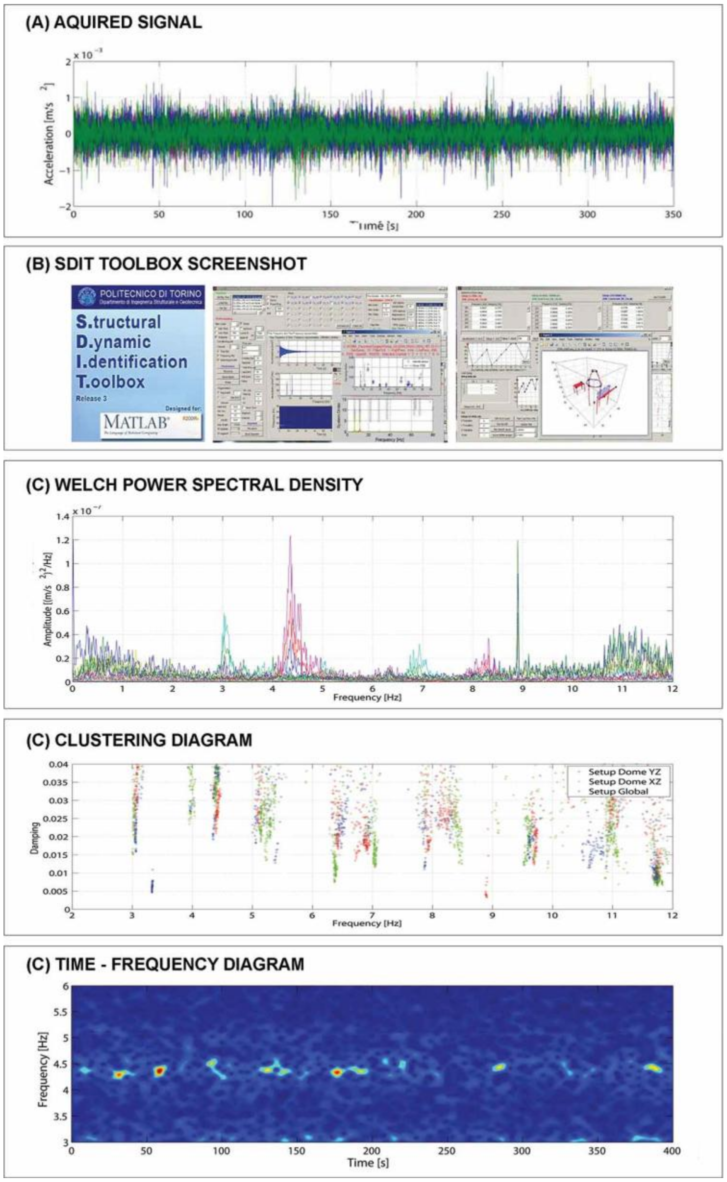


Figure 12. (c) Typical output in the identification process, (d) The four principal mode identified, in the case of "S. Caterina".

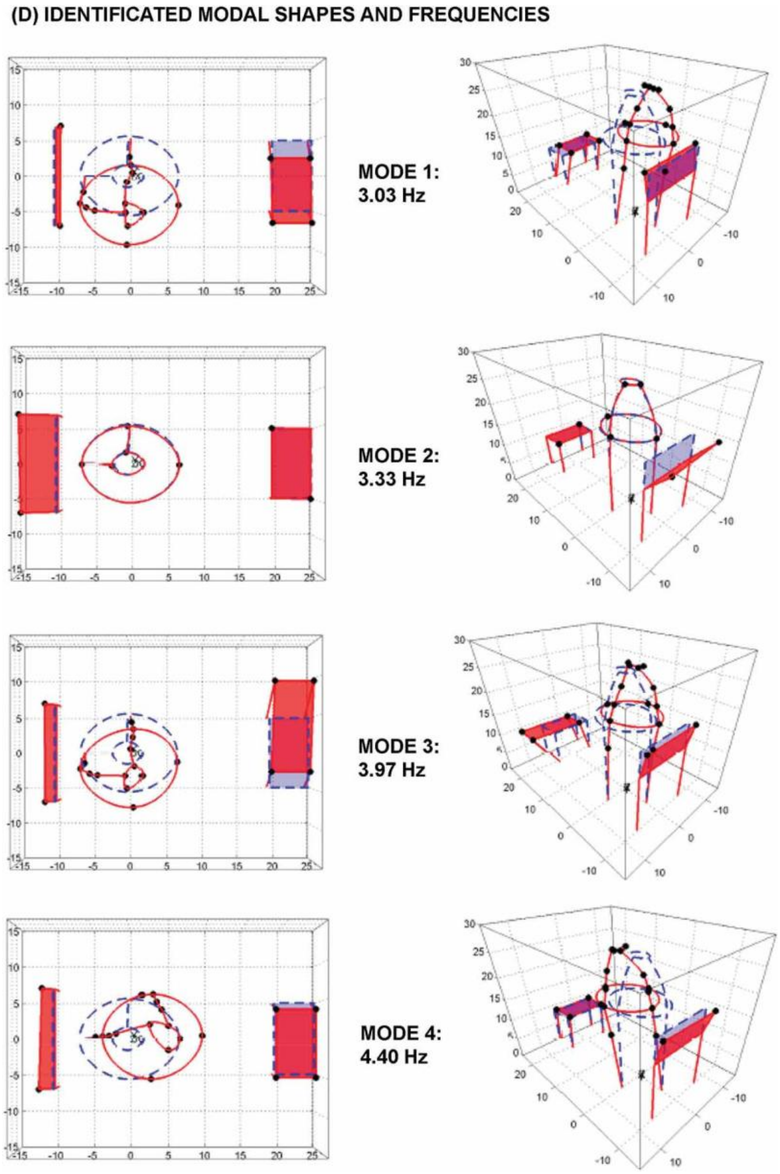


Figure 13. Two modes in the updated model of the Fossano Bell tower which highlight low resistance of the masonry in the middle part of the building.

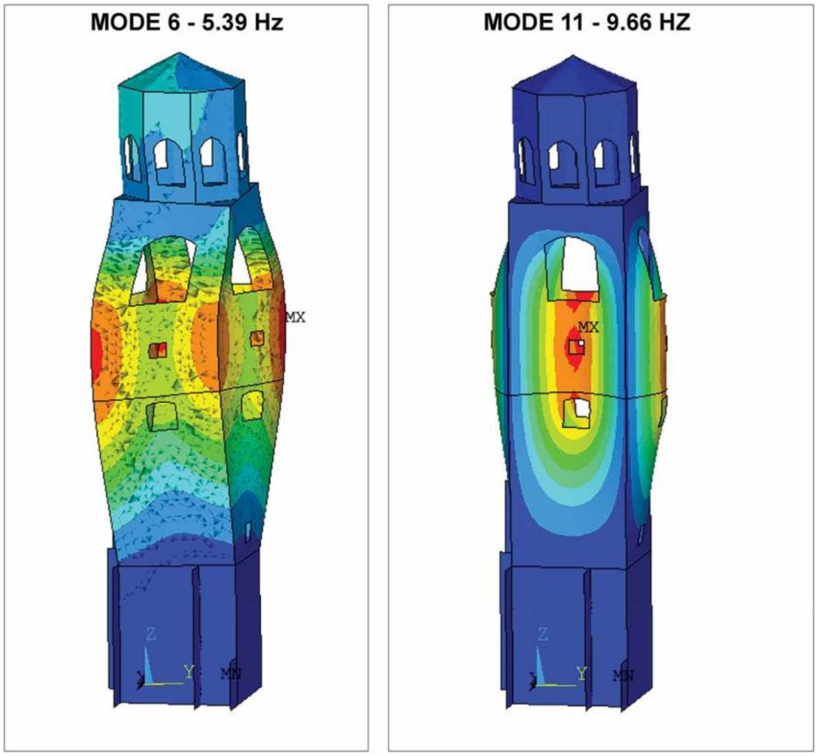
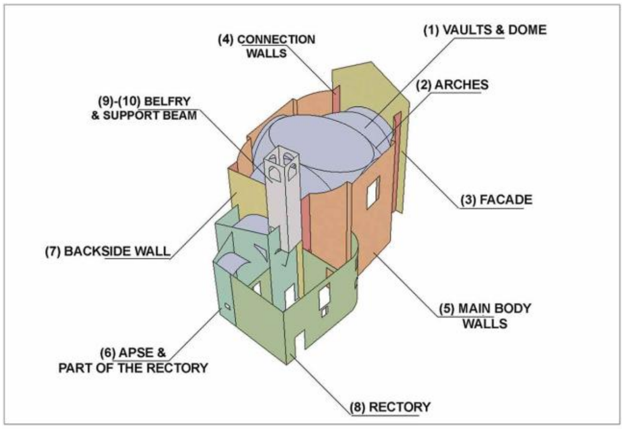


Figure 14. The subdivision in macro elements as it has been chosen for the "Madonnina della Neve". elements



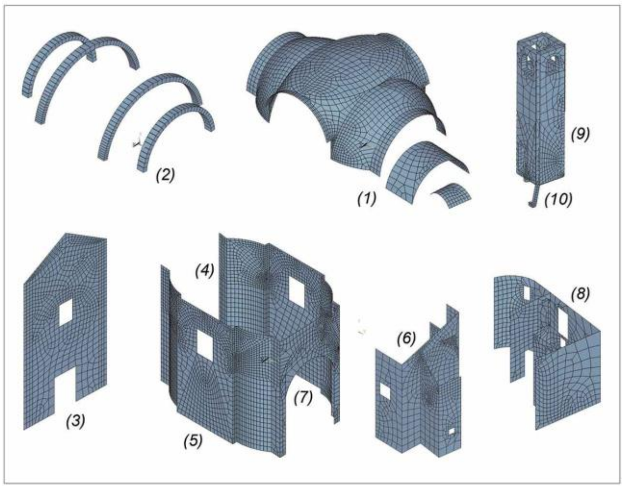


Figure 15. Model updating process performed on the "Madonnina della Neve" FE model

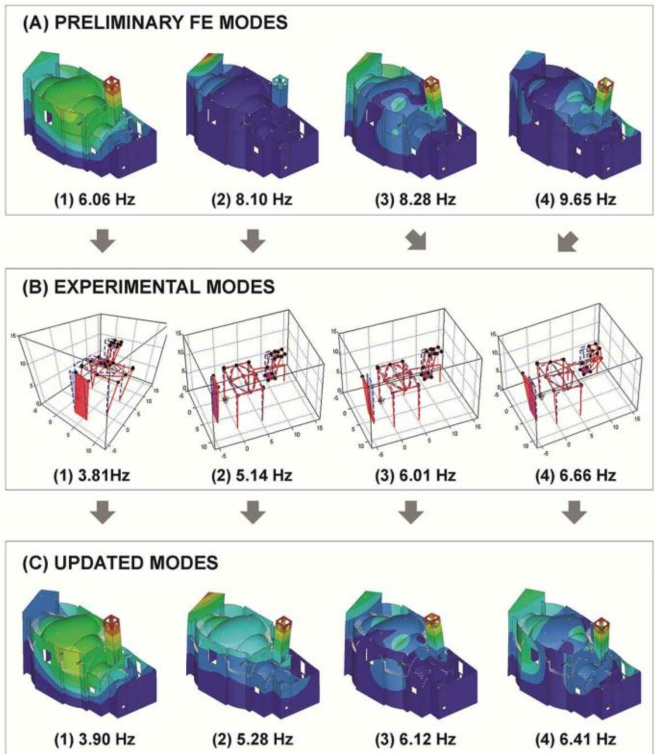


Figure 16. Young modulus distribution downstream the optimization; on the left the "Madonnina della Neve" and on the right the Fossano Bell Tower.

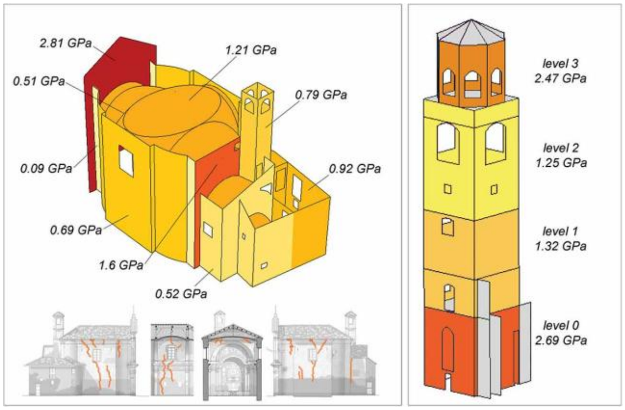


Figure 17. Compressive strengths of masonry at different levels (with reference to Figure 16) of Fossano Bell tower, coming from a core drilling campaign. On the right, core samples and cavities are showed for test 1 and 4.

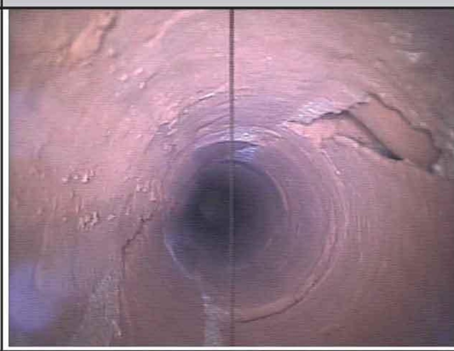
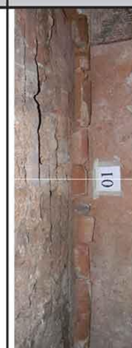
FOSSANO BELL TOWER			
LEVEL	COMPRESSIVE STRENGTH [MPa]		
0	2.00		
1	0.90		
2	0.90		

Figure 18. Displacement results of "S. Caterina": a) spectrum analysis and b) time-history analysis in both direction

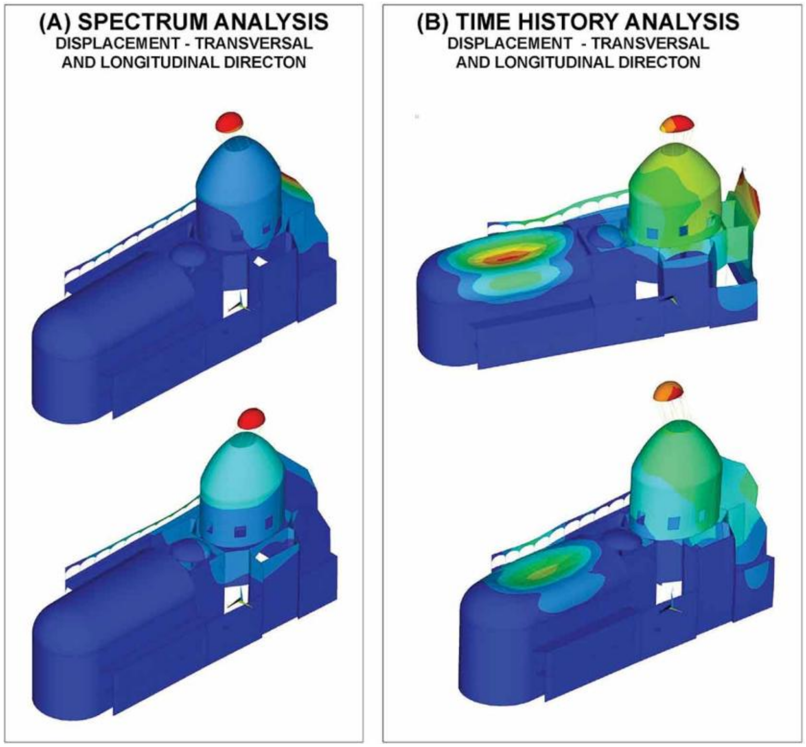


Table 1. Dynamic identification results in terms of frequencies and damping coefficients.

S. CATERINA			FOSSANO BELL TOWER			MADONNINA DELLA NEVE		
MODE N°	FREQ [Hz]	DAMP [%]	MODE N°	FREQ [Hz]	DAMP [%]	MODE N°	FREQ [Hz]	DAMP [%]
1	3.03	1.93	1	1.29	0.90	1	3.81	1.22
2	3.33	0.63	2	1.34	1.11	2	5.14	2.07
3	3.97	4.23	3	3.28	0.76	3	6.01	2.13
4	4.40	3.17	4	4.11	1.01	4	6.66	1.32
5	5.11	3.14	5	4.74	0.25	5	7.05	2
6	5.39	0.86	6	5.39	1.09	6	7.95	1.7
-	-	-	7	6.65	0.96	7	7.98	1.54
-	-	-	8	7.07	1.88	8	8.83	0.54
-	-	-	9	7.39	1.41	9	10.93	2.01
-	-	-	10	8.50	1.16	10	12.92	1.26
-	-	-	11	9.66	1.06	11	14.46	1.84
-	-	-	-	-	-	12	17.71	1.91

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Table 2. Results of the model updating process for the three examples: on the left, the MAC between the experimental modal shapes and the modal shapes determined by modal analysis of the FE updated model; on the right, comparison of the experimental frequencies with the frequencies of the FE updated model (Δf indicates the relative error).

SANTA CATERINA								
MAC – Identified modes vs Updated FEM modes					FREQUENCIES – Identified vs Updated FEM			
ID. MODE 4	0.10	0.03	0.00	0.91	MODE N°	FREQ FEM [Hz]	FREQ ID [Hz]	ΔF [%]
ID. MODE 3	0.91	0.03	0.63	0.01	1	2.94	3.03	2.9%
ID. MODE 2	0.01	0.99	0.02	0.00	2	3.53	3.33	6.0%
ID. MODE 1	0.90	0.00	0.59	0.02	3	4.27	3.98	7.2%
	FEM MODE 1	FEM MODE 2	FEM MODE 3	FEM MODE 4	4	4.33	4.4	1.6%
FOSSANO BELL TOWER								
MAC – Identified modes vs Updated FEM modes					FREQUENCIES – Identified vs Updated FEM			
ID. MODE 3	0.10	0.02	0.93	-	MODE N°	FREQ FEM [Hz]	FREQ ID [Hz]	ΔF [%]
ID. MODE 2	0.04	0.95	0.02	-	1	1.28	1.29	0.8%
ID. MODE 1	0.95	0.04	0.10	-	2	1.33	1.34	0.7%
	FEM MODE 1	FEM MODE 2	FEM MODE 3	-	3	3.35	3.28	2.1%
MADONNINA DELLA NEVE								
MAC – Identified modes vs Updated FEM modes					FREQUENCIES – Identified vs Updated FEM			
ID. MODE 4	0.02	0.15	0.16	0.78	MODE N°	FREQ FEM [Hz]	FREQ ID [Hz]	ΔF [%]
ID. MODE 3	0.05	0.13	0.85	0.35	1	3.90	3.81	2.3%
ID. MODE 2	0.05	0.83	0.57	0.31	2	5.28	5.14	2.8%
ID. MODE 1	0.91	0.04	0.08	0.00	3	6.12	6.01	1.9%
	FEM	FEM	FEM	FEM	4	6.41	6.66	3.6%

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	MODE 1	MODE 2	MODE 3	MODE 4	

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