



Ambient vibration test and numerical investigation on the St. Giuliano church in Poggio Picenze (L'Aquila, Italy)

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Received: 10 December 2018 / Accepted: 17 July 2019 / Published online: 30 July 2019
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Abstract

The paper deals with the investigation of the seismic behaviour of a masonry church sited in Poggio Picenze (district of L'Aquila) and damaged by the L'Aquila seismic event occurred in Italy on 2009 April 6th. This earthquake, classified as an exceptional event, caused significant damages to about 15,000 buildings. Above all, many of the cultural sites of L'Aquila and the surrounding villages, including churches, palaces and other monuments dating from the Middle Ages and Renaissance, were harmed in a severe way or demolished. The current work is framed in the post-seismic scenario of Poggio Picenze, a small town located about 10 km to the South-East of L'Aquila, and it is aimed at the seismic response assessment of the St. Giuliano church by means of experimental dynamic testing and numerical analyses. Firstly, an experimental campaign based on Ambient Vibration Tests (AVTs), that are non-destructive tests especially applied to historical and monumental buildings for evaluating their modal response, has been performed on the selected construction in the framework of the COST C26 Action as a cooperation activity between the Ss. Cyril and Methodius University of Skopje and the University of Naples "Federico II". As a result, the method has allowed to obtain the dynamic properties of the examined structure, namely natural frequencies, vibration mode shapes and damping coefficients. The experimental results have been subsequently used to implement a numerical FEM model of the church in the ABAQUS non linear code environment. Linear FEM analysis has been carried out to calibrate the experimental results and a suitable investigation in the nonlinear field has been conducted to predict the post-seismic structural behaviour of the church. Finally, the performed analyses have been of a fundamental importance for both detecting damages into the construction and programming effective retrofitting interventions.

Keywords Monumental constructions · Masonry structure · Seismic vulnerability · Ambient vibration test · FEM model

1 Introduction

Constructions may be classified as historical when they become part of our built heritage. Naturally, this does not necessarily mean that a historical building has a monumental character. Historical buildings carry their cultural significance attached not only to their formal architectural language, but also to their specific structural features, applied materials and building techniques and, being old, they also represent a part of the human life. Therefore, scientists have to put their knowledge at the service of culture, to both respect the historic value of the architectural heritage and guarantee appropriate safety levels, changing as less as possible the original design.

Monumental constructions are unique buildings having great architectural and artistic values and are characterized by their own unique history. Consequently, monuments, i.e. palaces, towers, castles, obelisks, theatres, churches,

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monasteries, abbeys, triumphal arcs, bridges, etc., represent the highest pieces of the cultural heritage of a country.

In Italy, the monumental heritage is particularly rich and the intent to preserve the historical heritage is very felt. Large areas of the Mediterranean area are unfortunately characterized by a high seismic hazard level and, in most cases, the vulnerability of ancient masonry constructions, monuments or historical centres is significantly high. In fact, it is well known that earthquakes have always represented the main cause of damage and losses to the cultural built heritage.

Thus, the necessity to protect such buildings is an important task for allowing them to survive destroying quakes without collapse and for safeguarding, at the same time, the human life. All ancient masonry buildings, indeed, were constructed following the rule of thumb, learning from the experience of previous similar structures. For this reason, as observed in the last Italian earthquakes, cultural heritage constructions are particularly vulnerable to dynamic actions, especially seismic loads.

The seismic analysis of monumental buildings belongs to a multidisciplinary study, where different information, such as construction history (year of erection, possible transformations, traumatic events), geometrical and structural critical survey, materials features and degradation and the detection of crack patterns, converges to each other. In this way, it is possible to diagnose the causes producing instability and/or degradation of structural elements, which can be classified as intrinsic and extrinsic types: the former are referred to the origin and nature of the monumental buildings and, therefore, to their vulnerability; while the latter are related to the site geographic conditions. Among these structures, there are also the Italian ecclesiastical buildings, having inestimable historical, artistic and cultural values, which were strongly compromised after the recent Italian seismic events [1–4].

The preliminary knowledge of their seismic performances is a pressing task to estimate a territorial level risk to make a priority ranking for planning further analyses. To this scope, it should be mentioned the vulnerability form developed by the Italian Civil Protection Department [5], used in the post-earthquake management phase to assign or not, through systematic surveys, the usability of church structures.

Recently, it has been proposed a new and simplified procedure, developed and validated at the University of Basilicata, to be applied also for ancient masonry churches, useful to evaluate the seismic risk at a territorial scale, including the study of threats which may affect the geographical context. This methodology, described in [6], validated in [7] and applied also for Chilean adobe churches in [8], is usefully applied to analyse the vulnerability and risk of a sample of churches under different hazard sources.

Alternatively, the recent trends on the detailed seismic vulnerability assessment of masonry churches focus on two crucial aspects, namely modelling and analysis issues on one hand and retrofitting interventions on the other hand.

It is common practice when performing seismic vulnerability assessment of masonry buildings to tackle separately out-of-plane and in-plane failure mechanisms of load-bearing walls. An enhanced knowledge of the dynamic behaviour of structures can play a relevant role in defining proper countermeasures for existing buildings and historical structures. The purpose of predicting the performance of buildings to different combinations of static and dynamic loads has attracted the interest of many researchers. Particularly, ambient vibration testing has become the main experimental method available for assessing the dynamic behaviour of full-scale cultural heritage structures, such as towers [9–16], monumental buildings [17–25] and bridges [26, 27]. This is especially true for churches where, since no excitation equipment is needed, a minimum interference with the normal use of the structure is needed [28–30]. These studies are at the base of different analysis methods on masonry structures in the nonlinear field, as detailed in [2, 4, 31, 32].

Unfortunately, it is not possible to reduce these construction typologies into standard static schemes; thus, the use of advanced Finite Element (FE) models is highly recommended. However, simplified models can be developed to give quantitative evaluation of the seismic vulnerability of structures, where limit analysis (LA) predominates [3, 33, 34]. Limit analysis combined with a macroelement approach is found to be a powerful tool, which can provide fast and reliable results [3].

Regarding the modelling process, it is worth mentioning the difficulty to deal with the masonry mechanical properties and to represent orthotropic behaviour within FE commercial codes.

In fact, even if micro-modelling studies are necessary to give a better understanding about the local behaviour of masonry structures, they require high computational efforts and, therefore, they are not commonly used in the professional practice.

On the other hand, a complete macro-model must reproduce an orthotropic material with different tensile and compressive strengths along the material axes, as well as a different inelastic behaviour for each material direction. A reduced number of orthotropic material models specific for masonry has been proposed in [35, 36] and a reasonable compromise is found considering a homogenization approach for the brick and mortar assemblage [37, 38].

With respect to the cultural heritage preservation, different retrofitting techniques are available and can be applied to reduce the seismic vulnerability. The traditional methodology consists on the introduction of consolidation systems for walls (i.e. steel prestressed tie rods) [39–43] and

horizontal structures (i.e. steel connectors for timber floors) [44]. Recently, great importance is given to the application of innovative materials, as Fiber Reinforced Polymers (FRP), that can be reversible and less invasive for architectural and cultural preservation [45–51].

The awareness of the importance of Italian cultural heritage and the desire to preserve the integrity of masonry churches under earthquakes have led towards the present study, where the seismic behaviour of a monumental church damaged by the 2009 L'Aquila seismic event has been experimentally and numerically investigated.

2 The L'Aquila seismic event and the historical centre of Poggio Picenze

Italy is one of the most seismic prone countries in the world. In particular, many active faults are located in Central Italy, where a 6.3 magnitude earthquake on the Moment Magnitude Scale (M_w), corresponding to both the 5.8 magnitude on the Richter Scale (M_L) and the VI–VII grade on the Modified Mercalli Intensity scale (MMI), occurred at 3.32 a.m. local time on 2009 April 6th [52]. Although the epicentre depth was not so deep (approximately 8 km), the seismic waves associated with shallow quakes produced very strong shaking and many damages and, furthermore, the main shock was followed by many aftershocks. The maximum Peak Ground Acceleration (PGA) recorded at a distance of about 5 km from the epicentre was estimated as 0.63 g and 0.42 g for the horizontal and the vertical earthquake components, respectively.

The damage distribution within the affected area was not uniform. In fact the heaviest damages were experienced in the centre of L'Aquila (VIII–IX MCS grade) and in some villages located in the middle Aterno valley, like Castelnuovo, Onna and Paganica (IX–X MCS grade). In total, 14 municipalities suffered an MCS intensity between VIII and IX, whereas those characterized by MCS intensity larger than VII were altogether 45 [53].

L'Aquila earthquake was considered as an exceptional event, since the maximum recorded acceleration within the epicentre area was larger than PGA of the elastic spectrum given by the Italian Code [30]. Moreover, it was a near-field quake, characterized by a considerable value of the vertical acceleration component. In fact, a near-field earthquake, being an impulsive motion and not a cyclic one, can be very dangerous; it was generally characterized by the following features [54]: (a) large pulses occurring at the beginning of the S waves motion with rather short duration; (b) motion pulse polarized in the direction normal to the fault strike; (c) large pulse amplitude both in horizontal and vertical directions. Consequently, the most important effects on

constructions with respect to far-field earthquake are as follows: (a) the superior modes are dominant in comparison to the fundamental ones; (b) the vertical component causes second-order effects, i.e. local or soft-floor mechanisms; (c) the structural ductility depends on the global stiffness rather than the soil condition.

Finally, local amplification effects, due to the particular soil structure, contributed to amplify seismic waves. In fact, L'Aquila was built on the bed of an ancient lake, characterized by a soil structure that amplifies seismic waves. In particular, the historic centre of L'Aquila is located on a fluvial terrace on the left bank of the Aterno River [55]. Therefore, the current geological setting of the L'Aquila basin results from a complex sequence of depositional events, due to erosion and tectonics. So, coupling near-fault conditions with site effects induced by the complex geological structures further contributes to the complexity of this earthquake ground motion.

The L'Aquila seismic event produced fatalities, injuries, homeless and severe damages to more than 10,000 buildings in L'Aquila and the surrounding area. Generally, the damaged buildings mainly belonged to the historical heritage and they were erected in the Roman times, in the Middle Age and in the Renaissance.

Starting from the days immediately after the seismic event, the Civil Protection Department members, in cooperation with a large number of Italian University researchers, visited those places in order to evaluate the usability of the whole built-up of L'Aquila and its districts [56, 57].

In May 2010, about one year after L'Aquila earthquake, an experimental research was performed in the framework of the COST Action C26 “Urban Habitat Constructions under Catastrophic Events” [58] as a cooperation activity between the Institute of Earthquake Engineering and Engineering Seismology (IZIIS) of the “Ss. Cyril and Methodius” University (Skopje, Republic of Macedonia) and the University of Naples “Federico II” (Naples, Italy) with the purpose of identifying the dynamic properties of a monumental church damaged by the seismic event in the historical centre of Poggio Picenze. This town, located about 10 km to the South-East of L'Aquila, is a small municipality with about 1000 inhabitants situated on the top of a hill, 760 m above sea level.

The historical centre of Poggio Picenze is the result of a continuous urban growth process from the ancient times up to the present days. In particular, the farming town can be divided into two different urban areas (Fig. 1).

The oldest nucleus was founded by Piceni around the 3rd century B.C. on the slope of Mount Picenze. The subsequent urban configuration developed around the medieval castle built approximately in the first century A.C. Originally, the ancient castle had fortified walls and six towers, including a high one in the middle. In this part, the urban planning is

Fig. 1 The historic centre of Poggio Picenze and the location of the St. Giuliano church



typical of a medieval town with buildings arranged in almost concentric arrays following the contours.

On the contrary, the other area has an irregular medieval urban shape with some important constructions, like the mercantile Medieval House, built in the thirteenth century.

The entire town suffered heavy damages during the 1762 October 6th earthquake, which required substantial reconstruction works. In fact, the castle of Poggio Picenze became unsafe and it was demolished. Ruins of this structure are still visible in the oldest part of the town.

In 2009, Poggio Picenze was one of the most damaged towns under the L'Aquila earthquake, which produced both significant damages to buildings of the historical centre and death of 5 people.

The historical centre consists of masonry building compounds generally developed on 2–3 storeys. Sack stone masonry with chaotic texture inside and bad quality mortar is the typical structure for load-bearing walls which are, in some cases, connected to each other by metal ties. However, building aggregates are characterized by regular ashlar walls, built with squared stone blocks cut to sizes corresponding to a set number of brickwork courses.

About horizontal structures, masonry vaulted ceilings largely covered the lower storey of the buildings, spanning along one or two directions. Other floor types with flexible

diaphragms are made of steel beams and vaulted or flat tiles. Instead, roofing structures are often composed of double frame timber beams with clay tile covering.

Other than building compounds, in Poggio Picenze there are several important monumental buildings, such as three churches (San Felice Martire, Visitazione and St. Giuliano) and two palaces (Galeota and Ferrari), which were experimentally investigated aiming at evaluating their dynamic properties for planning appropriate upgrading and retrofitting interventions [30].

In the current paper, the attention is devoted to the St. Giuliano church, which belongs to an aggregate of constructions and, therefore, has a dynamic behaviour conditioned from the presence of adjacent structures. In the following sections, the applied testing procedure and the main obtained experimental results are presented and discussed. The achieved numerical results, obtained by means of the ABAQUS numerical code in order to assess the building seismic behaviour, will be used to design an adequate retrofitting project of the church.

3 The St. Giuliano church

3.1 Historical and technical issues

The construction of the St. Giuliano church (Fig. 2) dates back to the early fifteenth century. The stone crest on the façade shows the date of 1589, where previously there was a small hospital remained in use until 1447.

The church has a single rectangular nave with timber beams covered by terracotta flooring of recent workmanship. The nave has a major altar and side altars of fine workmanship with newsstands of different shapes. Externally the church has a plastered façade with a horizontal

crowning defined laterally by pilasters. The church is accessed by a double ramp through a ground portal with stone upholstery having a triangular gable supported by knees brackets. On the façade, immediately above the portal, there is a square window with a broken triangular gable which is flanked by stone emblems. Currently, the church is within a masonry building compound, as shown in Figs. 3 and 4, where the geometrical features of the constructions are depicted. The remaining part of the original hospital consists of two rooms: one at the ground floor with vaulted ceilings and the other at the mezzanine floor.

The main vertical structure of the church consists of sack local stone masonry walls surmounted by a timber pitched roof (Fig. 4b). The walls, which have a thickness

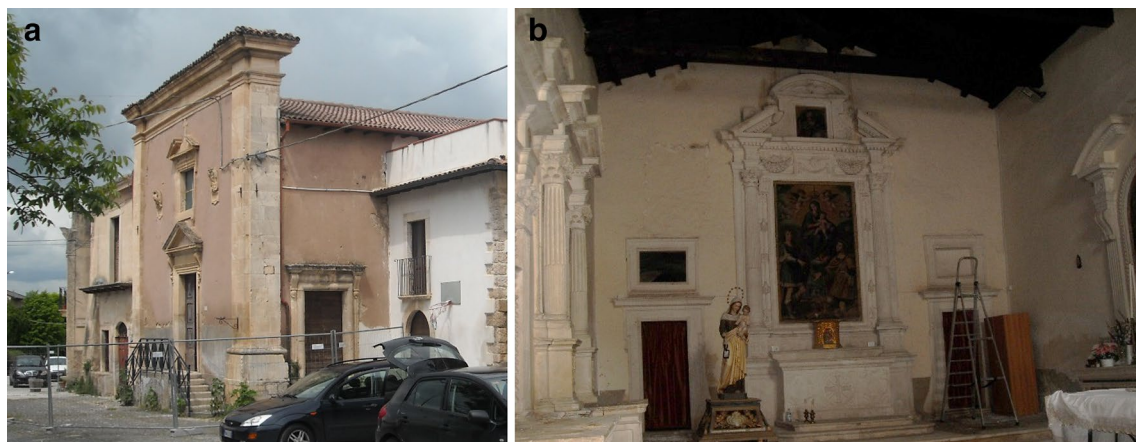


Fig. 2 St. Giuliano church: internal (a) and external (b) views

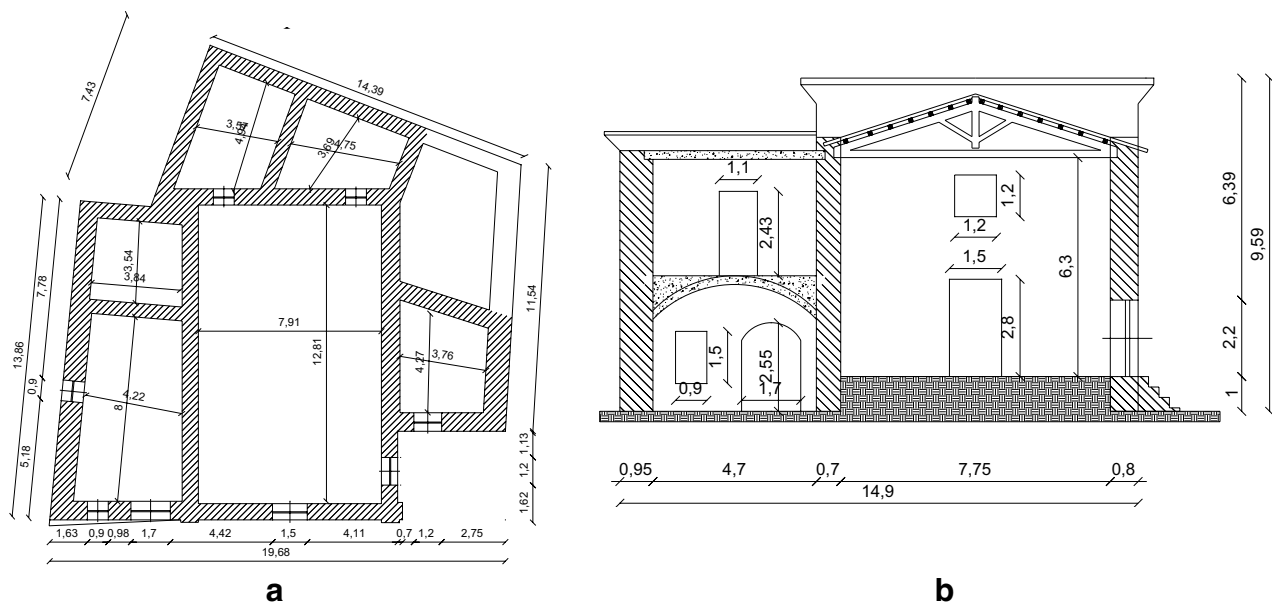
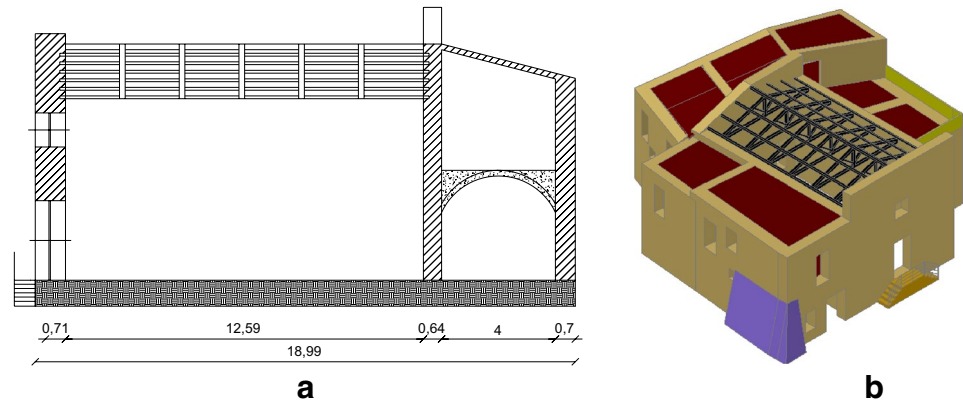


Fig. 3 Geometrical drawings of the St. Giuliano church: plan layout (a) and transverse section (b)

Fig. 4 Geometrical features of the St. Giuliano church: longitudinal section (a) and 3D drawing of the entire building aggregate (b)



of 70 cm, are covered by the lime mortar plaster. The main façade contains elegant architectural decorations typical of the Romanesque—Aquilano style. Due to the earthquake, this façade showed an out-of-plane overturning mechanism, which produced some cracks in the intersection zones with longitudinal walls. In addition, shear and vertical in-plane cracks have been detected on the façade wall. Another damage also concerns the church longitudinal wall on the right side of the entrance, which has suffered a horizontal bending mechanism triggered by the thrust of the first level vaults of the neighbouring building. Finally, some minor cracks into arches and vaults have been detected. The pictures of the main damages identified after earthquake are reported in Fig. 5.

Parameters that have influenced the described failure mechanisms of the masonry wall are certainly the high slenderness, its deficient boundary conditions and low resistances of both bricks and mortar joints.

3.2 Experimental test

The definition of the health state of buildings based on their dynamic characteristics should be performed by means of experimental in situ testing, which can be performed through Ambient Vibration Testing (AVT) methods. The AVT is a non-destructive analysis technique which is very useful for buildings having historical and artistic importance. This test is generally to be preferred for historic structures, since no excitation equipment is needed. Nowadays, environmental excitations are always present and, therefore, this test type implies a minimum interference with the normal use of the structure. In fact, structures are usually excited by wind, traffic and human activities and the measurements should be taken for a long duration to ensure that all modes of interest are sufficiently activated. Thus, ambient vibration testing has recently become the main available experimental method

Fig. 5 Incipient overturning of the main façade (a) and horizontal bending mechanism of one of the longitudinal walls (b) of the church



to evaluate the dynamic behaviour of full-scale structures. In this test, typology natural frequencies, corresponding modal shapes and damping coefficients represent the main properties to be monitored and acquired.

The experimental test herein illustrated is referred to the investigation conducted on the St. Giuliano church to measure the dynamic response of the construction under excitations associated to environmental actions. The experimental activity was performed to extract modal parameters, which were successively processed and utilized for the implementation of an FEM numerical model by means of a specific structural analysis software.

It is important to highlight that the measurements were performed only on the church structure, which represents one of the structural units of the masonry aggregate constituting the original hospital.

The instrumentations used by the IZIIS laboratory members to carry out the ambient vibration test included the following equipments: three SS-1 Ranger seismometers (Fig. 6a), kinemetrics product for ambient vibration measurements and a four-channel signal conditioner for filtering and amplifying the measured signals (Fig. 6b).

The Ranger seismometers are widely recognized as excellent short-period field instruments. They have small size, high sensitivity, adjustable natural period and rugged water-tight construction. In addition, these tools can be used as vertical or horizontal seismometers by simple adjustment of the mass centring spring. The used seismometer type is a “moving coil” style acceleration transducer. The coil is stationary, however, while the strong permanent magnet serves as the seismic inertial mass. The mass is supported and constrained by annular springs at the top and bottom of the moving magnet. In vertical position, the unit, which has a weight of about 5 kg, has a diameter of 305 mm and a height of 140 mm. The basic natural period is of 0.35 s, but it is extended to 1 s or more by means of small rod magnets that surround the periphery of the mass, interacting with its magnetic field.

For amplification and simultaneous control of the seismometers, the signal conditioning model SC-1, also manufactured by Kinemetrics, was used. Four channels of ambient vibrations are recorded to allow for satisfactory resolution in low-frequency regions of the power spectra and the transfer functions. This four-channel frequency analyzer covers a frequency range from 0.02 to 25.6 Hz and allows for a complete frequency analysis of the signals obtaining both amplitude and phase spectra. In particular, the data sets consisted of records of velocity signals with duration of 1000 s and the sampling frequency was 200 Hz.

The ambient vibration testing procedure herein used consists of real-time recording of vibrations and processing of records. The initial test was the dynamic calibration one. During this test, all sensors (seismometers, accelerometers) were placed on the same position in the same direction, the signals were recorded simultaneously and Fourier spectra were obtained. The resonant frequencies of the structure can be preliminarily defined by a dynamic calibration test, but final definition of the natural frequencies is possible after obtaining the vibration mode shapes. After this calibration test, the sensors are placed at different levels and different points of the structure, but in the same direction, for simultaneous recording, so to achieve the vibration mode shapes.

In the examined case, the seismometers measured the vibration signals recorded in different point of the structure, normalized with respect to a specific steady point, called Reference Point (RP), where the dynamic calibration test was performed. In particular, since in ambient vibration tests the excitation is unknown, the modal amplitude corresponding to the RP in each setup and for each resonant frequency is used to correlate the partial mode shape results obtained in the different setups. The recording duration was set as equal to 1000 s that is an enough long time to eliminate the influence of possible non-stochastic excitations, which may occur during the test.

Afterwards, the amplified and filtered signals from the seismometers were collected by a high-speed data

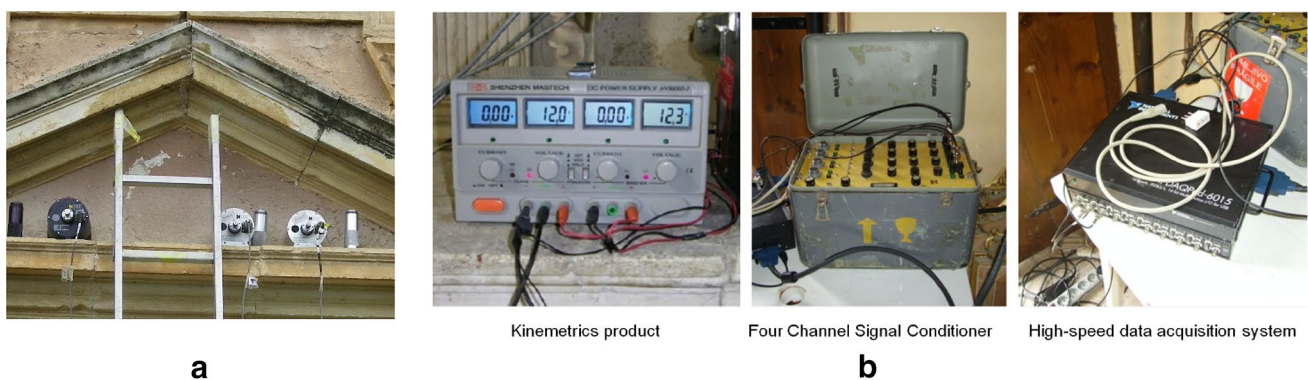


Fig. 6 The used seismometers (ranger type) (a) and the equipment for AVT (b)

acquisition system, which transforms the analogue signals into digital ones. PC and special software for online data processing were used to plot the time histories of the recorded velocities together with the Fourier amplitude spectra (FAS) of the response at each measured point.

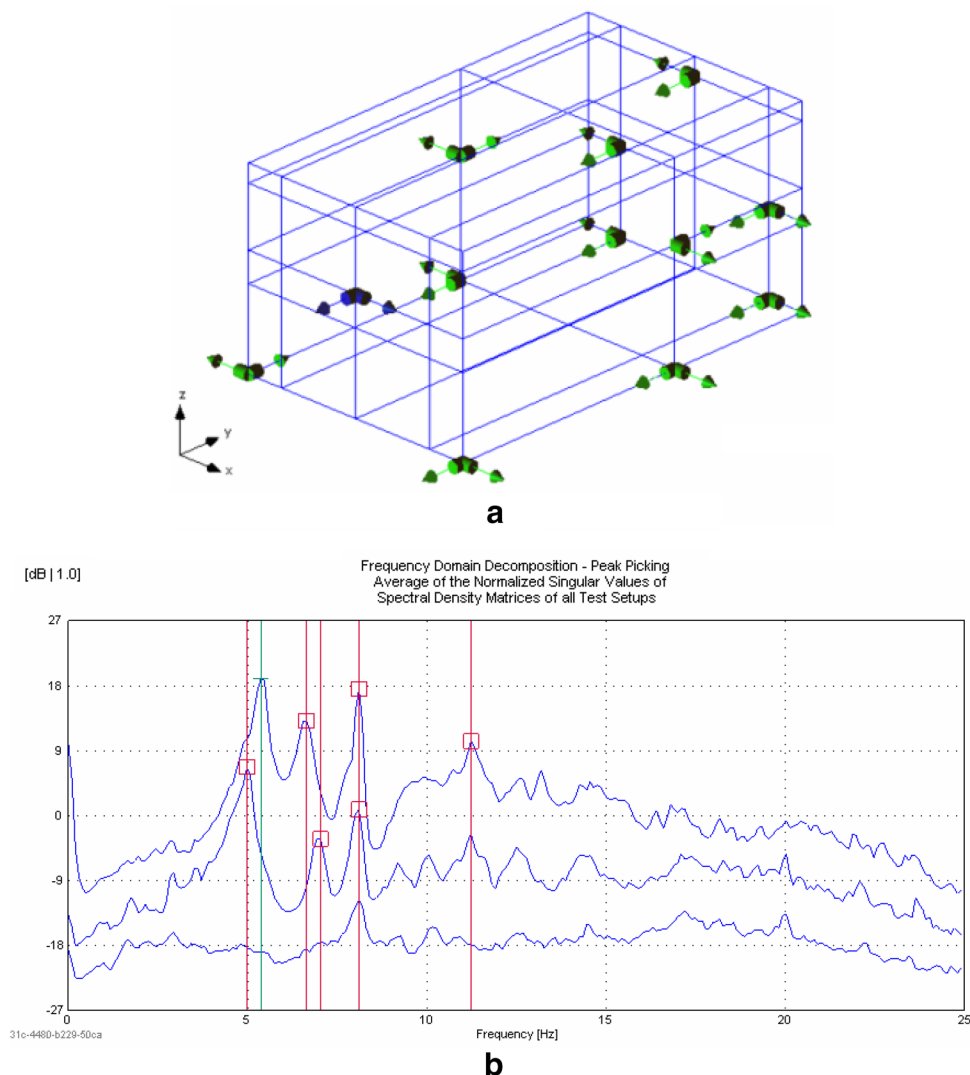
Finally, for post-processing and analysis of the recorded vibrations at all measuring points, the ARTeMIS software, which is a powerful and versatile tool designed for Operational Modal, Experimental Modal, Operating Deflection Shapes and Structural Health Monitoring analyses, was used [59]. Measurements channels have to be linked to a geometry node and mode shapes and operating deflection shapes need a realistic test geometry for proper animation. The geometry generator produces a realistic test geometry and the data organizer is capable of administrating multiple test setups, where the sensors are moved over the structure from measurement to measurement, keeping few sensors in fixed positions as references. In particular, ARTeMIS is able to handle multiple test setups (rowing sensors) and multiple

reference points for increased mode shape accuracy [59]. In this software, the natural frequencies and the vibration mode shapes are determined using the Peak Picking and the Frequency Domain Decomposition (FDD) techniques [60, 61].

The structure of St. Giuliano church was measured in 11 different points to assess its dynamic properties. Measurement points were placed at different positions along the church height, by exploiting the presence of altars, gables and openings, in order to reconstruct the modal vibration shapes as precise as possible. They are indicated in green in Fig. 7a, where also the reference point (RP) with blue colour is highlighted. Four vibration modes, corresponding to the main modal vibration shapes related to the actual damage state of the construction, were identified from the analysis of the complete set of data representative of the entire construction.

The peak-picking of dominating frequencies in the obtained spectrum is depicted in Fig. 7b, where it is apparent that only the three lower peaks on the first SV line (SV1)

Fig. 7 Test setup (a) and dominant frequencies (b)



have been “picked”, while the other peaks (on SV1 and SV2) can be conceivably associated with local modes.

As shown in Fig. 8, the vibration mode shapes are characterized by the main frequencies of 5.37 Hz and 6.64 Hz, where the structure moves in longitudinal direction with dominant separation of the façade wall, and the frequency of 7.03 Hz, in which the torsion effect is noticeable as a result of the influence of the neighbouring connected structures, which give rise to a building compound characterized by in-plane and in-elevation structural irregularities. Moreover, in the previously mentioned torsion vibration mode, it is also noticed the horizontal bending mechanism of the longitudinal masonry wall detected after the occurred 2009 earthquake.

All the experimental data are summarized in Table 1, where frequencies obtained from the FDD method and damping coefficients associated with each vibration mode are shown. It is worth to precise that in the dynamic analysis procedures, the damping coefficient, based on both the system ability to absorb dynamic energy and on the duration of vibration modes, provides a realistic motion attenuation. In fact, dynamic analysis results are generally influenced by damping ratios, which have been estimated using the Enhanced FDD, based on free decays corresponding to identified spectral peaks. In this specific case, the damping values are found to be variable from 1.9 to 2.2%.

In the following analysis phase, an FEM numerical model of the monumental building has been implemented, being calibrated on the basis of the above-mentioned experimental results.

3.3 FEM analysis

A numerical activity has followed the experimental campaign to better investigate the dynamic response of the St. Giuliano church. It is important to precise that the model

Table 1 Dominant frequencies and damping coefficients

Mode shape	Dominant frequency (Hz)	Damping coefficient (%)
Longitudinal	5.37	2.2
Longitudinal	6.64	2.2
Torsion	7.03	1.9

of the entire aggregate has been herein generated to both take into account the influence of other aggregate units on the church seismic behaviour and simulate with accurate reliability the experimental results.

The structural finite element (FE) model of the masonry structural compound has been implemented with the ABAQUS/CAE computer code [62], used to perform specific numerical frequency analyses. This program provides a simple and consistent interface for creating, submitting, monitoring and evaluating results from numerical simulations.

In particular, the model has been generated by importing in the FE program a three-dimensional solid model of the construction created in a computer-aided design program. To properly assess the structural interaction among the different constitutive parts, the geometrical model accurately reproduces all the main components of the building, including openings and floors.

More in detail, the numerical model has been calibrated according to the following basic steps:

1. Selection of the finite element types for different building parts;
2. Calibration of the elastic mechanical properties of materials;
3. Calibration of the mesh size.

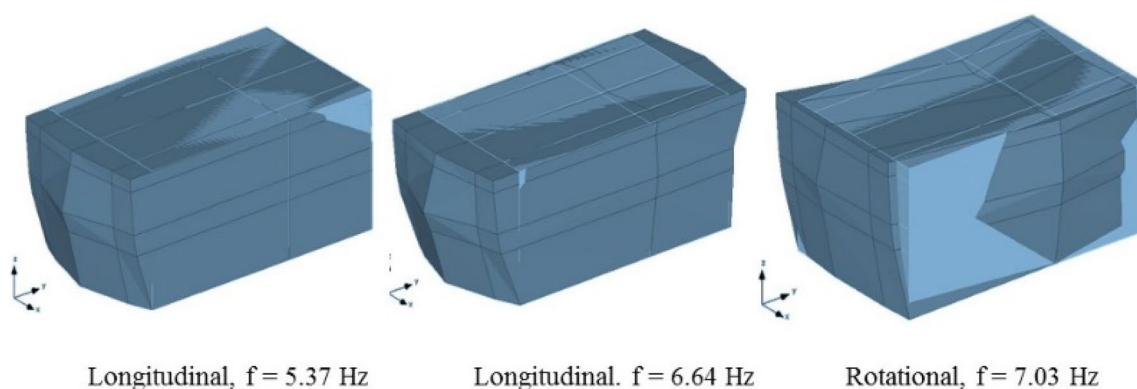
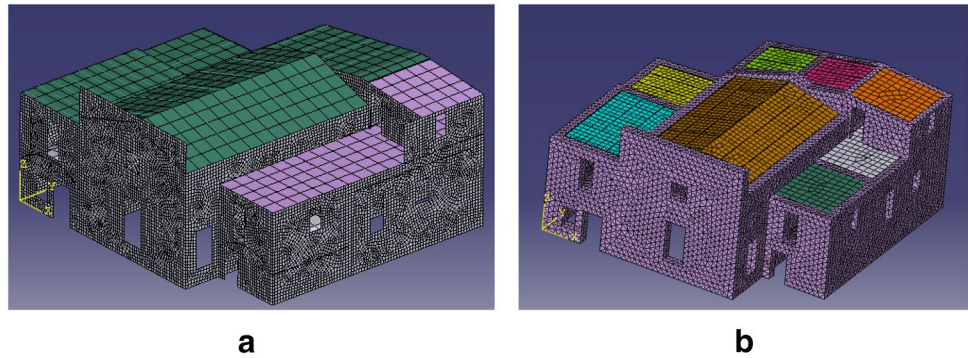


Fig. 8 Experimental mode vibration shapes

Fig. 9 The ABAQUS FEM model: with brick and shell elements (a), with brick elements only (b)



About the choice of finite element types (step 1), the examined masonry structure has been partitioned by means of tetrahedral 3D brick (C3D4, 4-node linear tetrahedron) elements. For roofing structural systems, instead, timber trusses and beams have been modelled with truss elements, while different techniques have been used to model the timber planking. In particular, in a first phase all the roofing planking components have been modelled as shell elements (Fig. 9a); subsequently, they have been modelled as brick elements (Fig. 9b).

The preliminary analyses have evidenced that the best model is that with all brick elements, since it does not consider the shell–brick interactions, which can lead to both solution convergence problems and significant analysis times. So, this FEM model based on brick elements only has been selected to perform subsequent analyses.

As far as the material modelling is concerned (step 2), a continuum homogeneous material has been assumed for masonry. In particular, aiming at identifying the global structural response in the frequency analyses, only the material density and elastic properties, i.e. Young modulus (E) and Poisson's coefficient (ν), are required. Therefore, since experimental data on mechanical characteristics of masonry were not available, material properties were derived from the Italian Technical Code [33], which provides a range of values for different masonry typologies.

Thus, for the examined masonry, which can be classified as rubble stones with an irregular texture, the variability of some mechanical properties has been considered. As a consequence, parametric analyses have been performed by modifying manually the Young modulus value of construction materials in the range provided by the Italian standard code [33, 63], taking into account their degradation state, in order to establish the best agreement between experimental results and numerical ones in terms of vibration frequencies. Definitely, the implemented FEM model consists of several parts having the mechanical properties listed in Table 2.

Subsequently, a mesh sensitivity analysis (step 3) has been carried out to refine the results obtained in the previous phase. By comparing the computational effort with the

Table 2 Mechanical properties of materials used in different FEM model parts

Part	Type of ABAQUS element	Material	Mechanical parameters		
			E (MPa)	ν	w (kN/m ³)
Facade	C3D4	Masonry	350	0.3	19
Lateral walls	C3D4	Masonry	350	0.3	19
Floors	C3D4	Concrete	20,000	0.2	25
Trusses, beams and planking	C3D4	Timber	9000	0.3	4.5

Table 3 Comparison among experimental and numerical frequency values

Mode	Experimental frequency (Hz)	Numerical frequency (Hz)
Longitudinal	5.37	5.30
Longitudinal	6.64	6.70
Torsion	7.03	7.20

result accuracy, it was found that a mesh size of 0.30 m is able to better approximate the experimental results.

Finally, by combining the selected elastic modules of materials and the mesh sizes, the numerical frequency analysis of the church was carried out and the achieved results were compared to the experimentally measured ones.

The numerical frequency analysis has shown two longitudinal modes ($f=5.30$ Hz and $f=6.70$ Hz) and a torsion mode ($f=7.20$ Hz) modes. In Table 3, it is apparent that there is a satisfactory agreement between numerical and experimental results in terms of both sequences of vibration modes and natural frequency values.

Finally, the numerical mode vibration shapes are depicted in Fig. 10. From this figure, it is apparent that the structure shows local effects, substantially represented by the out-of-plane overturning mechanism of the main façade. The attained damage, caused by the lack of effective connections

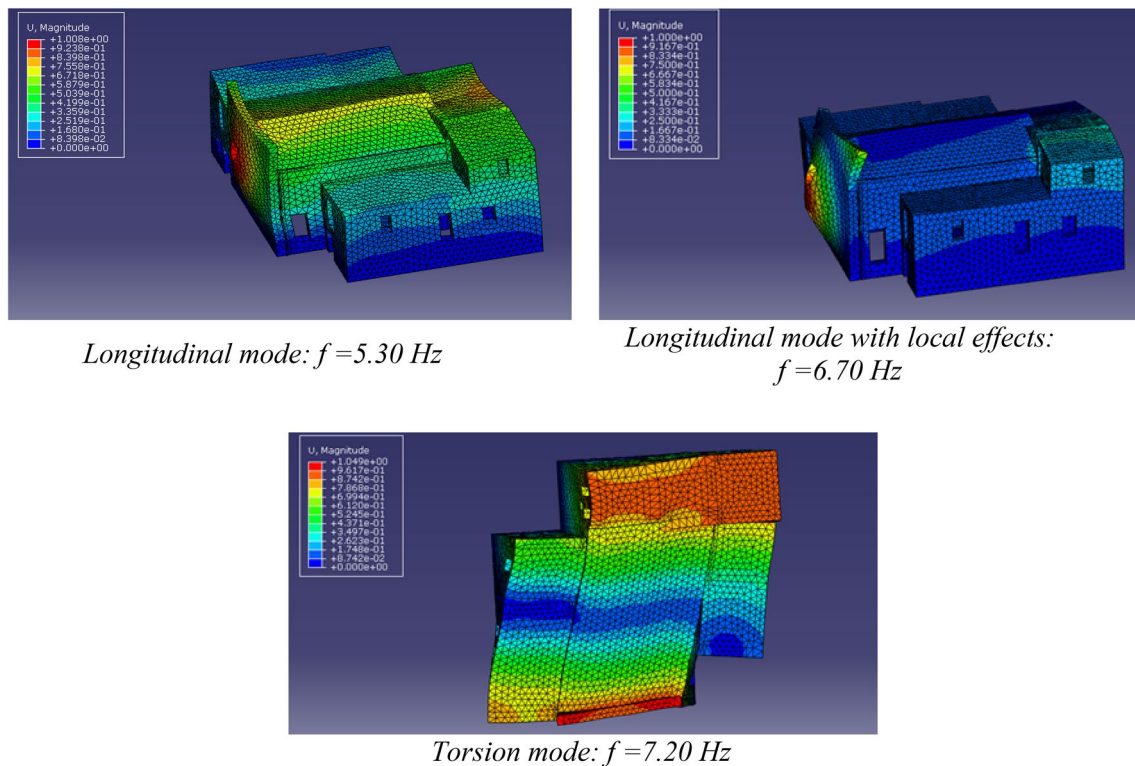


Fig. 10 Numerical modal vibration shapes

between the main façade and longitudinal masonry walls, modelled in the ABAQUS program through the “contact” constraint based on a tangential behaviour with friction coefficient of 0.40, is analogous to what verified under the 2009 earthquake (see Fig. 5a). In addition, according to the torsion vibration mode of the church ($f = 7.03$ Hz) depicted in Fig. 8, it is apparent that, as occurred under the L’Aquila seismic event (see Fig. 5b), the horizontal bending mechanism of the longitudinal wall on the right side of the entrance occurs. However, even if the numerical modal vibration shapes are not able to reproduce all the occurred crack patterns, which were due to the combination of other concurrent factors (i.e. effect of vertical accelerations neglected in this study, influence of other aggregated structural units only roughly modelled, etc.), since the main damages detected under earthquake have been also attained from the numerical point of view, the reliability of the calibrated FE model can be confirmed.

4 Conclusive remarks

In the present paper, the dynamic in situ testing of a monumental church within a masonry building compound located in Poggio Picense has been performed with the purpose to obtain its dynamic features, namely natural frequencies,

mode shapes and damping coefficients, after the 2009 L’Aquila earthquake.

The experimental test results have shown that in the frequency range provided by the used four-channel analyzer several frequencies are of interest. In fact, due to the heavy damage state and stiffness degradation of the church, the spectra also contain the frequencies of the damaged parts, like front (façade) walls, vaults and arches, which complicate the identification of the global structural frequencies. Therefore, under the frequencies of 5.37 Hz and 6.64 Hz the structure moves in the longitudinal direction, with dominant separation of the façade wall, while for frequency of 7.03 Hz the torsion effect is noticeable as a result of the influence of the neighbouring connected structures. Moreover, equivalent damping coefficients values ranging from 1.9 to 2.2% have been experimentally determined.

The experimental dynamic results have been used for numerical investigation of the seismic response of the St. Giuliano church. To obtain a good agreement between experimental and numerical results, the model of the entire masonry aggregate has been implemented. Therefore, a church FEM model has been set up by means of the ABAQUS nonlinear numerical code by calibrating adequately the elastic properties of masonry (350 MPa) and the structure mesh size (0.30 m) according to the experimental test results. The implemented FEM model has provided a

good agreement of results in terms of both experimental natural frequencies and mode vibration shapes.

Finally, the numerical simulation of experimental tests herein illustrated has been of a fundamental importance for detecting the damages into the construction and thus, as a further study development, for programming future correct retrofitting interventions, whose effectiveness will be evaluated by new ambient vibration measurements to be carried out after repairing and strengthening operations.

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