EXPERIMENTAL AND NUMERICAL INVESTIGATION ON THE VISITAZIONE CHURCH IN POGGIO PICENZE (AQ)

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ABSTRACT

In recent time, an earthquake shocked L'Aquila, the capital of the Abruzzo Region, and the surrounding areas. This seismic event, generated by a normal fault, occurred on April 6th, 2009 at 3:32 a.m. (local time) and was produced by a maximum vertical dislocation of 25 cm at a hypocentre depth of about 8.8 km.

L'Aquila and the surrounding districts suffered significant damages to historic buildings. In particular, the historic centre of L'Aquila was partially destroyed. Also, the so called "minor" architecture, consisting of the surroundings small historic centres, were grievously damaged.

In May 2010, about one year after L'Aquila earthquake, an experimental study was performed in the framework of the COST Action C26 "Urban Habitat Constructions under Catastrophic Events" (http://www.civ.uth.gr/cost-c26) 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" with the purpose to identify the dynamic properties of a monumental church in the historical centres of Poggio Picenze, a district of L'Aquila, damaged by earthquake.

In the paper, the applied testing procedure and the main obtained experimental results are presented and discussed. The achieved numerical results, gotten by means of the ABAQUS numerical code in order to assess the building seismic behaviour, will be used to design an adequate retrofitting project for the church.

Keywords**:** L'Aquila earthquake, cultural heritage, environmental vibration tests, modal frequencies, FEM model

1. INTRODUCTORY REMARKS

Constructions may be classified as historical when they become part of our built heritage. Naturally, this does not necessarily mean that a historical building is necessarily of a monumental type.

Historical buildings, which are parts representative of the human life, carry their cultural significance attached not only to their formal architectural language but also to their specific structural features, applied materials and building techniques.

On the other hand, monumental constructions are unique buildings having a great architectural and artistic values and characterised by their own exclusive history. Therefore, monuments represents the highest pieces of the cultural heritage of a Nation, they including churches, palaces, towers, castles, abbeys, triumphal arches, bridges, etc.

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

In recent times, an earthquake shocked L’Aquila, the capital of the Abruzzo Region, and the surrounding areas. This seismic event, generated by a normal fault (Fig. 1a), occurred on April 6th, 2009 at 3:32 a.m. (local time) and was produced by a maximum vertical dislocation of 25 cm at a hypocentre depth of about 8.8 km (Fanale et al., 2009). This event took the form of an earthquake swarm, which is a typical feature of central Apennine seismicity. The seismic sequence included, in fact, hundreds of events located in real time by the seismologists at INGV (*Italian National Institute of Geophysics and Volcanology*) monitoring room, using data from both the National seismic network and the satellite emergency network installed in the epicentral area (Fig. 1b).

The mainshock was rated 5.8 on the Richter Scale (ML) and 6.3 on the Moment Magnitude Scale (MW). Furthermore, three large aftershocks (ML= 4.8, ML= 4.7 and ML= 5.3) occurred on April 7th in the towns of Onna, Fossa, and Paganica (South-East of L’Aquila). The event was the strongest among a sequence of 23 earthquakes having M_W greater than 4 and occurred between 2009 March, 30th and 2009 April, 23rd (Fig. 2a), it providing strong motion recordings from accelerometer stations placed very close (4-5 Km) to the epicentre. The damage distribution within the affected area was not uniform. In fact the heaviest damages were experienced in the centre of L’Aquila (VIII $\div$ 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 a MCS intensity between VIII and IX, whereas those characterized by MCS intensity larger than VII were altogether 45 (Galli and Camassi, 2009).

L’Aquila earthquake was an *exceptional* event, since the maximum recorded acceleration within the epicentral area was larger than PGA of the elastic spectra given by the Italian Code (M. D.,

2008) (Fig. 2b). Furthermore, it was a *near-field quake*, characterised by a considerable value of the vertical acceleration component. Finally, local amplification effects, due to the particular soil structure, contributed to amplify seismic waves.

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 earthquake occurred when most people were sleeping. So, a large number of people were killed (305) or injured (1.500). The fatalities were concentrated in two age groups, namely 20-29 years and over-70 years, but this did not reflect the demographic age of L'Aquila province. In fact, the peak in the 20-29 years group was due to the collapse of a student hall in the downtown of L'Aquila. Moreover, the earthquake produced the temporary evacuation of 70000-80000 residents and 24000 of them remained without home (AA.VV., 2009).

The whole population of the towns listed in the official earthquake damage declaration was 60352. Generally, towns were composed of people in the range [1000 ÷ 3000], with only two larger municipalities having 5000 and 8500 inhabitants.

After the earthquake, about 10000 – 15000 buildings were destroyed or damaged. Above all, many of the region cultural sites, including Romanesque churches, palaces and other monuments dating from the Middle Ages and Renaissance, were harmed in a severe way or demolished. The total damage was estimated larger than 25 billion €.

L'Aquila and the surrounding districts suffered significant damages to historic buildings. In particular, the historic centre of L'Aquila, which in English means "The Eagle", was partially destroyed. The built up heritage of this city is represented by churches and monuments, which include the Fountain of the Ninety-Nine Spouts, the 16th century Spanish Castle, the Basilica of St. Bernardino, the Church of St. Massimo, the Church of St. Mary in Collemaggio and the Government Palace (Fig. 3).

Also the so called "minor" architecture, consisting of the surroundings small historic centres, e.g. Fossa, Onna, Paganica, Castelnuovo and Poggio Picenze, the latter representing the town where the case study of this work is located, were grievously damaged.

Starting from the days immediately after the seismic event, the Civil Defense Department members, in cooperation with a large number of Italian University Institutions researchers, visited those places in order to evaluate the usability of the whole built-up of L'Aquila and its districts (Indirli et al., 2012).

In the following months, for housing management, the emergency was handled by the Civil Protection Department. In particular, two types of dwelling were provided for the homeless people (Fig. 4): apartments of the CASE Project (*Anti-Seismic, Sustainable and Environment- Friendly buildings*), and wooden cabins of the MAP Plan (*Temporary Accommodation in Modular Housing Unit*).

In May 2010, about one year after L’Aquila earthquake, an experimental study was performed in the framework of the COST Action C26 “Urban Habitat Constructions under Catastrophic Events” (http://www.civ.uth.gr/cost-c26) 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” with the purpose to identify the dynamic properties of a monumental church in the historical centres of Poggio Picenze (AQ) damaged by earthquake.

In the following sections, the applied testing procedure and the main obtained experimental results are presented and discussed. The achieved numerical results, gotten by means of the ABAQUS numerical code in order to assess the building seismic behaviour, will be used to design an adequate retrofitting project for the church.

2. POGGIO PICENZE

Poggio Picenze is a small town situated on the top of a hill, 760 meters above sea level, and it is located about 10 km to the South-East of L’Aquila. It lies along a slope located at the left (north) side of the river Aterno valley. The municipality has a population of about 1000 inhabitants.

The historical centre is the result of the process of continuous urban growth from the ancient times up to the present days. In particular, the farming town can be divided into two different urban areas (Fig. 5).

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 1st century A.C. Originally, the ancient castle had fortified walls and six towers, including a high one in the middle. Therefore, in the oldest part, the urban planning is typical of a medieval town with buildings arranged in almost concentric arrays which follow the contours. On the contrary, the other area, which is the new one, has an irregular urban plan with some important palaces, like the mercantile Medieval House, built in the 13th 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 (Galeota, 2006).

Nowadays, the historical centre consists of masonry complex, generally ranging from 2 to 3 stories. Sack 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. In general, the first level horizontal structure consists of vaulted floors, while the other levels are constituted by either wooden or steel floors. The most common roof typology is the pitched one. Moreover, from the architectural viewpoint, finishing, doorways, balconies, patios and porches are usually embellished with local limestone, the so-called white stone of Poggio Picenze, which has a gentle appearance and is easy to work.

Poggio Picenze was one of the most damaged towns under the Abruzzo mainshock with a grade of 5.8 and 6.3 on the Richter scale and the moment magnitude one, respectively. Also, several thousands of aftershocks, more than thirty with a Richter intensity greater than 3.5, occurred.
Consequences of 2009 L’Aquila earthquake were dramatic, since 5 people died and significant damages to buildings of the historical centre were recorded (Fig. 6).

3. THE VISITAZIONE CHURCH

The *Visitazione* church, located in the historic centre of Poggio Picenze, may be considered one of the most important monumental building of the town, because of its historic, artistic and sacred value. This small chapel (Fig. 7) was built probably between the 14th and 15th century and it was enlarged in 1832, as declared within an internal biblical epigraph.
The church has a rectangular shape with a 18 m long and 6 m large unique nave (Fig. 8). The building exhibits a local stone masonry structure surmounted by a wooden pitched roof covered by clay tiles, probably rebuilt in 1980 after the collapse of the original one. In particular, the roofing main structure consists of cherry-wood trusses (200 mm x 200mm cross-section) and purlins (100mm x 100mm cross-section). Internal views of the church are illustrated in Figure 9.
The main façade contains elegant architectural decorations typical of the Romanesque- Aquilano style. Due to the earthquake, this façade shown an out-of-plane overturning mechanism, actually prevented by an appropriate retaining steel system (Fig. 7), which produced heavy cracks between longitudinal walls and the same façade wall. Also, shear and vertical in plane cracks have been perceived on the façade wall, they being graphically reported in Figure 10.

4. THE EXPERIMENTAL ACTIVITY

4.1 General

In order to evaluate the seismic behaviour and response of historical buildings, several important issues should be considered. One significant issue is the estimation of earthquake ground motion based on amplitudes, as well as on the frequency content of both local and far seismic sources, also considering local soil conditions. Other important factors influencing seismic response of buildings are strength and deformability characteristics of materials, as well as the interaction between the local soil and the structure. Furthermore, the dynamic properties of the structure, namely natural (resonant) frequencies, mode shapes and damping capacity, should be considered also as other main aspects.
Therefore, the definition of a building actual state in terms of its dynamic characteristics should be performed by means of experimental in-situ testing, by applying either ambient vibration testing methods or force based ones.

The Ambient Vibration Test (AVT) is a non-destructive test very useful for building having a historical and artistic importance. This test is generally to be preferred for testing a historic structure, because no excitation equipment is needed. Actually, environmental excitations are always present and, therefore, the test implies a minimum interference with the normal use of the structure. In fact, the structure is usually excited by wind, traffic and human activity and the measurements are taken for a long duration to ensure that all modes of interest are sufficiently activated. Thus, ambient vibration testing has recently become the main experimental method available to evaluate the dynamic behaviour of full-scale structures, generally represented by the following properties: natural frequencies, corresponding modal shapes and damping coefficients.

The experimental test was conducted on the *Visitazione* church in May 2010 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 utilised for the implementation of a FEM numerical model by means of a specific structural analysis software.

The instrumentations used by the IZIIS laboratory to carry out ambient vibration test includes the following equipments: three Ranger type seismometers (Fig. 11) and a Kinematics product for ambient vibration measurements and a Four Channel Signal Conditioner for filtering and amplifying the measured signals (Fig. 12).

The seismometers (Fig. 11) measure the vibration signals recorded in different point of the structure. Since the input – output correlation is not a priori noted, a steady point must be fixed as a Reference Point (RP), in order to normalise each measured point as respect to the RP amplification and, therefore, to identify the global dynamic response. Afterwards, the amplified and filtered signals from the seismometers is collected by a high-speed data 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.

More precisely, the Fast Fourier Transform (FFT), obtained for each measured point (P_i) is simultaneously compared with the RP recorded response. This latter is constantly monitored during the vibration test in order to determine a transform function $H(\omega)$, which constitutes an intrinsic function of the structure. Thus, the following ratio is used to define the transform function:

$$H(\omega) = \frac{FFT_{Pij}(\omega)}{FFT_{RPj}(\omega)} \qquad (1)$$

where:

- *i* is the spatial position of the i-th seismometer;
- *j* is the testing direction (X or Y).

Finally, for post-processing and analysis of the recorded vibrations at all measuring points, the ARTeMIS software is used. In this software the natural frequencies and the vibration mode shapes can be determined by using the Peak Picking and the Frequency Domain Decomposition (FDD) techniques.

The employed software ARTeMIS is very good for graphical presentation of the obtained data (Krstevska et al., 2008). In fact, the operational modal analysis made with this computer program is geometry driven. 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. The drawing is object orientated enabling design of complex structures using more basic sub-elements, each with its own grid plane and coordinate system. 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. If multiple test setups are used, the data organizer automatically identify the reference sensors (http://www.svibs.com/products/ARTeMIS_Testor.aspx).

The measurements on the Visitazione Church were conducted in 11 different points of the structure, which were monitored into transversal and longitudinal directions, so enabling to achieve longitudinal, transverse and rotational frequencies and mode shapes (Krstevska et al., 2010).

The measured points are highlighted in green on the geometry of the monument generated by ARTeMIS software (Fig. 13a). The reference sensor was placed on the main facade, it being reported with a blue filled circle in Figure 13b.

4.2 Test results

Six vibration modes were identified from the analysis of the complete set of data representative of the entire building. The peak-picking of dominating frequencies in the obtained spectrum is depicted in Figure 14.

The vibration modal shapes are presented for the transversal frequency (f=3.51Hz), for the torsion frequency (f=5.08Hz) and for the longitudinal one (f=7.32Hz). The shapes of the fundamental modes are displayed in Figure 15.

All the experimental results are summarised in Table 1, where frequencies obtained from the FDD method and damping coefficients associated to each vibration mode are shown. It is worth to precise that in the dynamic analysis procedures, the damping coefficient provides a realistic motion attenuation. This ratio is based on the material damping properties. Dynamic analysis results are generally influenced by the damping ratio. This parameter is based on both the system ability to absorb dynamic energy and on the duration of vibration modes. In this specific case, the damping range is variable from 1,5% to 3,3.%.

In the following analysis phase, a FEM numerical model of the monumental building has been implemented, it being calibrated on the basis of the obtained mentioned experimental results.

5. THE NUMERICAL ACTIVITY

A numerical activity has followed the experimental campaign in order to better investigate the dynamic response of the *Visitazione* church. The structural Finite Element (FE) model of the church has been implemented by means of specific numerical frequency analyses implemented in the framework of the ABAQUS/CAE computer code. This program provides a simple, 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. In order 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 the floor.

The numerical model has been calibrated according to the following basic steps:

1. selection of the finite element types;
2. calibration of the elastic mechanical properties of materials;
3. calibration of the mesh size.

According to the first step, the examined masonry structure has been discretised by means of tetrahedral 3D brick (C3D4, 4-node linear tetrahedron) elements (HKS, 2004). For roofing structural system, instead, different techniques have been used to model the wooden trusses and the timber floor. In particular, in a first phase all the roofing elements have been modelled as brick elements (Fig. 16a); subsequently, wooden trusses and the timber floor have been modelled as beam and shell elements, respectively (Fig. 16b).

The aforesaid preliminary analyses have evidenced that model with brick elements only provides the most reliable results in terms of damage detected into the structure after earthquake (Fornaro, 2011). So, this FEM model have been selected in order to perform seismic 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 frequency analyses, only density and elastic properties of material, such as Young modulus (E) and Poisson's coefficient (v), are required. Therefore, since experimental data on mechanical characteristics of masonry were not available, material properties have been derived from the New Technical Italian Code (M. D., 2008), 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 (Fig. 17), the variability of some mechanical properties listed in Table 2 has been considered. In this table, f_m and τ_0 are the masonry mean compressive strength and shear one, respectively; E and G are the masonry Young modulus and shear modulus, respectively, and w is the material density. In particular, the elastic modulus (E) has been opportunely changed for calibrating the FEM model according to the experimental evidences by considering, according to the Italian Code (M. D., 2008), the following values:

- E = 345 MPa (half of the code minimum value);
- E = 525 MPa (half of the code maximum value);
- E = 690 MPa (code minimum value);
- E = 870 MPa (code average value);
- E = 1050 MPa (code maximum value).

As a consequence, parametric analyses have been performed by modifying the E value in order to establish a good agreement between experimental results and numerical ones.

Later on, a mesh sensitivity analysis (step 3) has been carried out in order to refine the results obtained in the previous phase. Generally, in macro model approaches, the mesh is usually generated in such a way that each element contains at least a portion of horizontal and vertical mortar joints surrounding the masonry unit. However, in the examined chaotic masonry, mortar joints and bricks cannot be univocally identified, as shown in Fig. 17.

Thus, the mesh size has been opportunely calibrated considering the following dimensions:

1. *Fine Mesh*: side length of 0,20 m;
2. *Medium Mesh*: side length of 0,30 m;
3. *Coarse Mesh*: side length of 0,40 m.

Finally, the last aspect of the model calibration dealt with the seismic damage of the structure, given by the overturning of the main façade. This mechanism, due to the lack of connections between orthogonal walls, has been considered in the modelling by means of the insertion of a very deformable (E = 1 MPa) masonry in the corner zones of the facade for a vertical strip of 2,5 cm.

Therefore, parametric frequency analyses have been performed combining different values of elastic modulus and mesh sizes in order to identify the natural frequencies and the vibration shapes corresponding to the experimentally measured ones.

All the main results obtained by parametric analyses in terms of measured numerical frequency values are shown in Table 3.

The comparison among results evidences that frequencies increase as the elastic modulus augments. This increment is substantially due to the increase of stiffness and, therefore, to the period reducing. In addition, frequency increases as the mesh size amplifies. As a consequence, the structural stiffness may be overestimated in case of fine size mesh.

Conclusively, by comparing the computational efforts with accuracy of results, it has been found that the *medium mesh* (0,30 m) is able to better approximate the experimental results.

After the model calibration, a masonry portion with better elastic properties has been added to the FEM model in order to take into consideration the presence of the overturning retaining system of the church main façade (see Fig. 13 b).

Definitively, the implemented FEM model consists of several parts having the mechanical properties listed in Table 4.

Afterwards, frequency analysis has been performed and natural frequencies have been found for the first 10 vibration modes. The frequencies of the first three fundamental modes are:

1. Transversal mode: f = 3,5 Hz
2. Rotational mode: f = 5,1 Hz
3. Longitudinal mode: f = 7,33 Hz

The fundamental modal shapes are depicted in Figures 18 and 19.

From these figures, it may be pointed out that the most deformable part of the structure is the main facade, as shown in Fig. 18, despite the presence of the retaining device.

6. COMPARISON AMONG PERFORMED INVESTIGATIONS

In Table 5 the numerical natural frequencies are compared with experimental ones. From comparison it is seen that there is a satisfactory agreement of result in terms of both sequence of vibration modes and natural frequency values.

In Figure 20 the comparison between experimental modal shapes and numerical ones is illustrated.

7. CONCLUSIVE REMARKS

The dynamic in-situ testing of a monumental church located in Poggio Picenze was performed with the purpose to obtain its dynamic characteristics (natural frequencies, mode shapes and damping coefficients) after the 2009 L'Aquila earthquake.

The test results have shown that in the range from 0 to 25 Hz 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, arches etc., which complicate the identification of the global structural frequencies, they assuming values of 3,51, 5,08 and 7,32 Hz for the transverse, torsional and longitudinal modes, respectively. Moreover, equivalent damping coefficients values ranging from 1,5% to 3,3% have been experimentally provided.

The experimental dynamic results have been used for numerical investigation of the seismic response of the *Visitazione* church. Therefore, a church FEM model has been implemented by means of the ABAQUS non linear numerical code by calibrating adequately the elastic properties of masonry in order to simulate experimental test results.

The set-up FEM model has given a good agreement of results in terms of both experimental natural frequencies and mode vibration shapes. So, in this phase, the numerical simulation of experimental tests has been of a fundamental importance for detecting the damages into the building and, in a next future, to program a correct retrofitting intervention.

Finally, as a further study development, it is recommended to repeat the ambient vibration measurements after repair and strengthening of the church in order to identify the effectiveness of the applied intervention.

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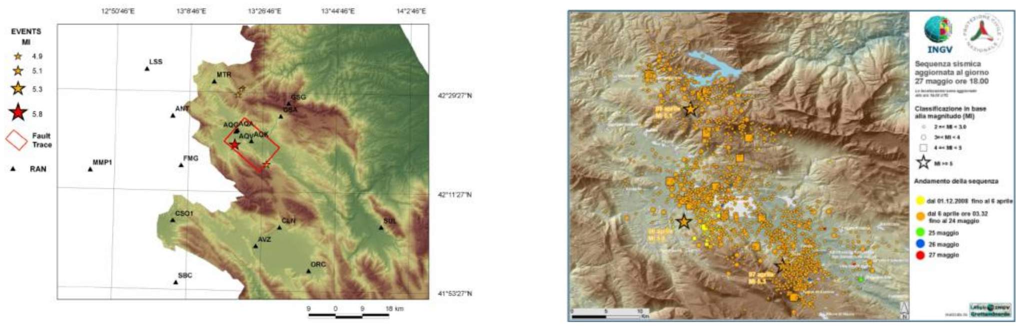
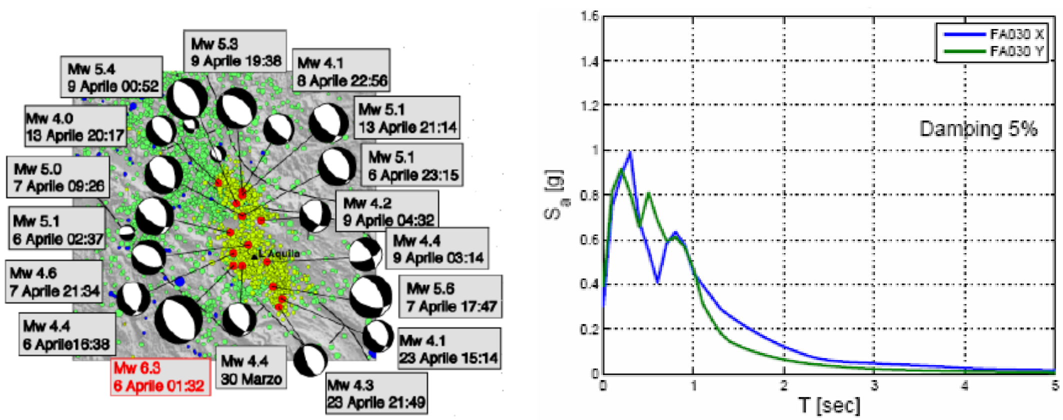


a) b)

Fig 1. L'Aquila earthquake: fault geometry (a) and seismic sequence (b)



a) b)

Fig 2. Sequence of seismic events occurred in the L'Aquila district (a) and elastic acceleration spectrum 4.3 Km far from the epicentre (b)

The Basilica of St. Bernardino

The Government Palace

Fig 3. Monumental buildings of L’Aquila after the earthquake

Building of the CASE Project

Wooden cabin in Onna

Fig 4. Types of dwellings provided for homeless people

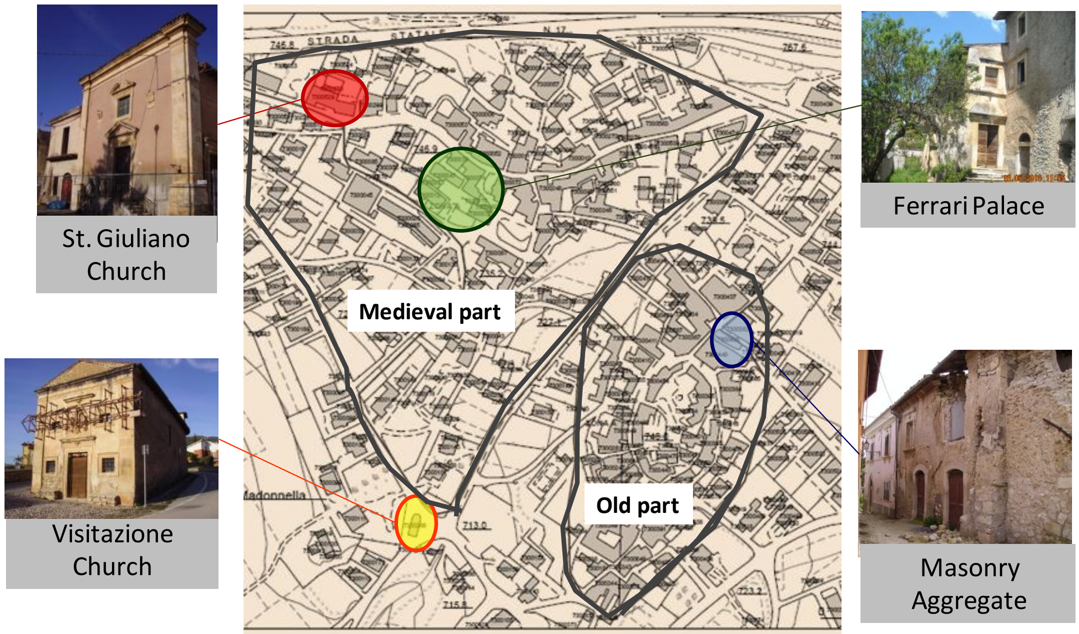


Fig 5. The historic centre of Poggio Picenze with the main monumental constructions

Fig 6. Damages occurred in the historic centre of Poggio Picenze after the 2009 L’Aquila earthquake

Fig 7. The *Visitazione* church: external views

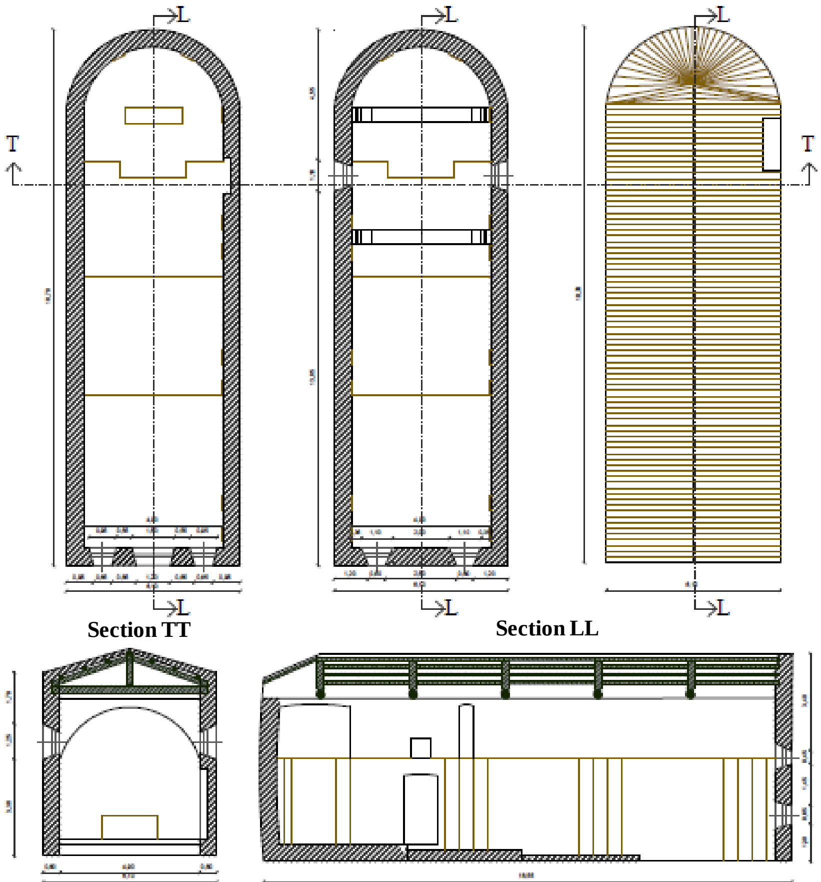
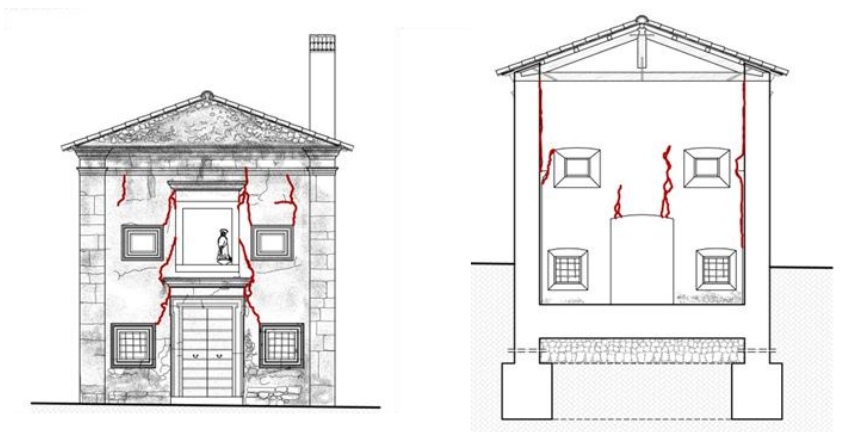


Fig 8. Geometrical features of *Visitazione* church

Fig 9. The *Visitazione* church: internal views



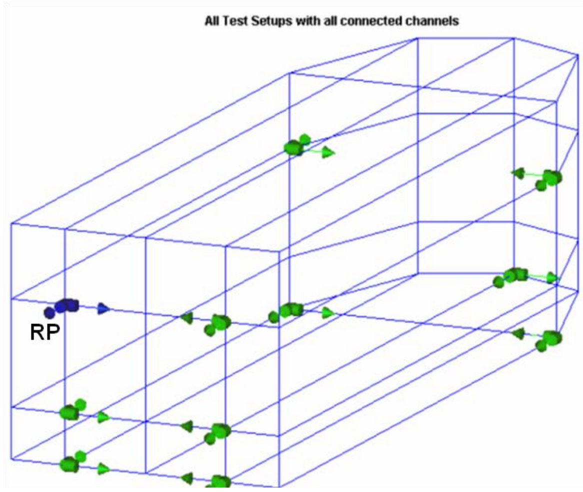
a) *b)*

Fig 10. External (a) and internal (b) views of the church main facade

Fig 11. The used seismometers (Ranger type)

Kinemetrics product Four Channel Signal Conditioner High-speed data acquisition system **Fig**

Fig 12. The test set-up for ambient vibration tests



a)

b)

Fig 13. Measurement points a) and reference point (in blue) (b) of the structure

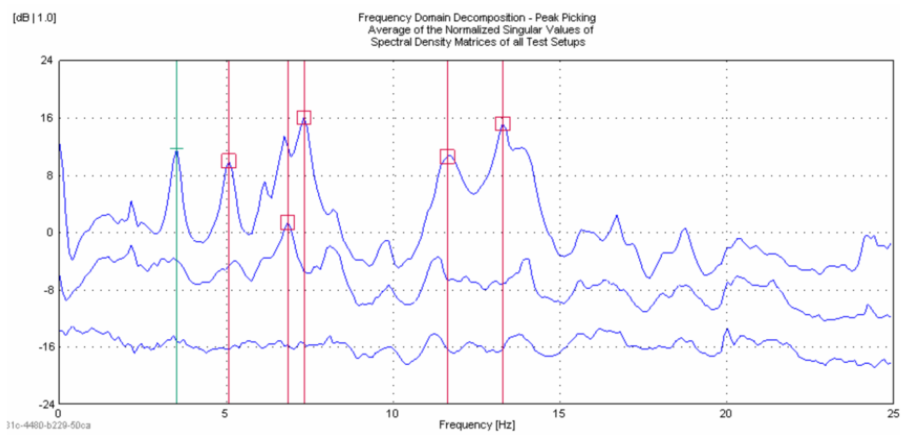
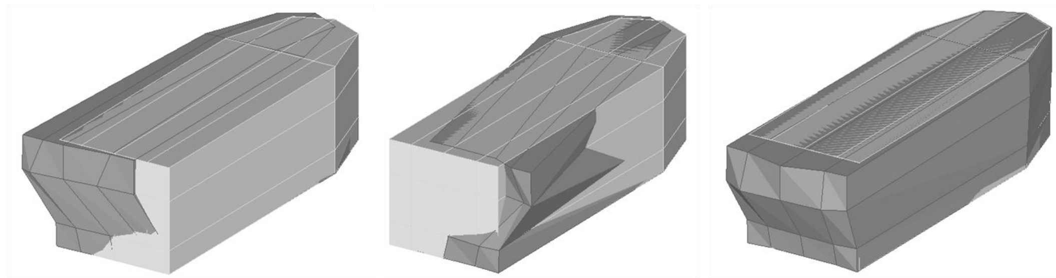
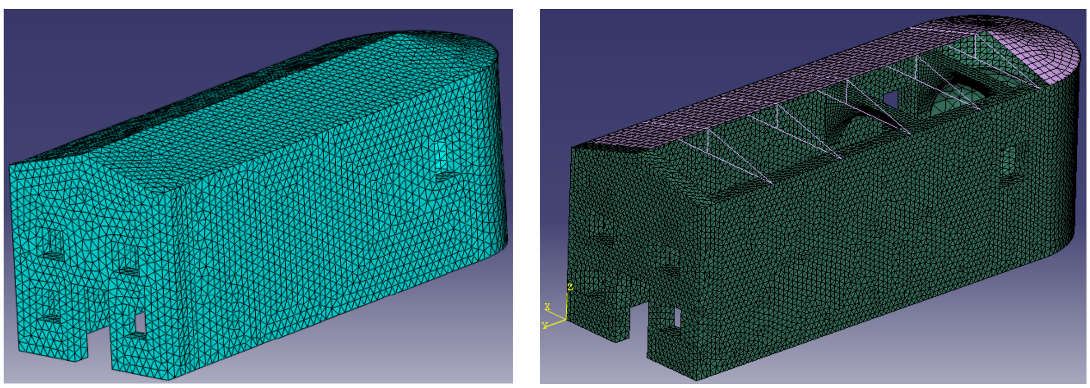


Fig 14. Peak-picking of the dominant frequencies



Transverse mode : f = 3.51Hz　　Rotational mode : f= 5.08Hz;　　Longitudinal mode : f = 7.32Hz;

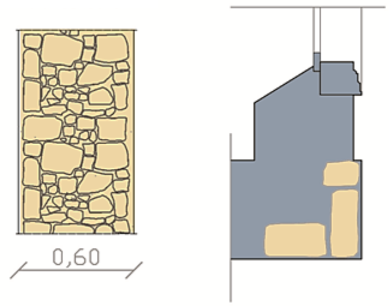
Fig 15. Vibration modal shapes



a)　　*b)*

Fig 16. The ABAQUS FEM Model of the *Visitazione* church:

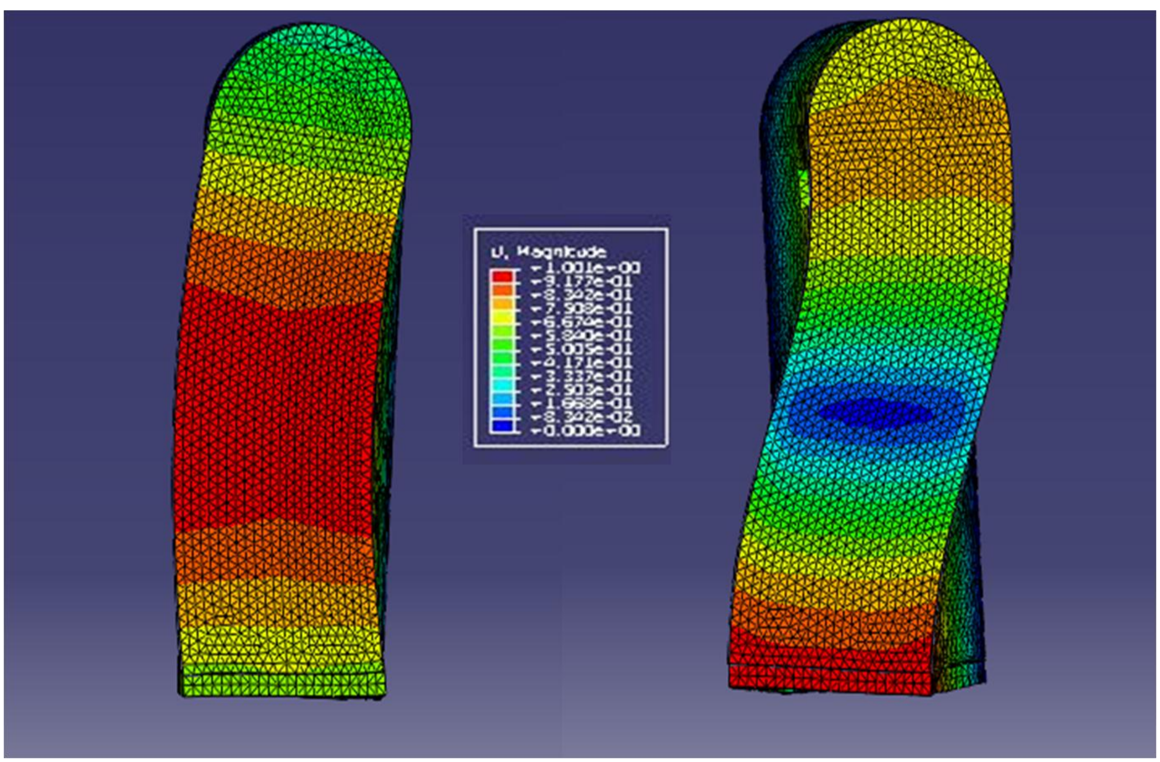
with brick elements only (a); with brick, shell and beam elements (b)



Calcareous Rubble Masonry

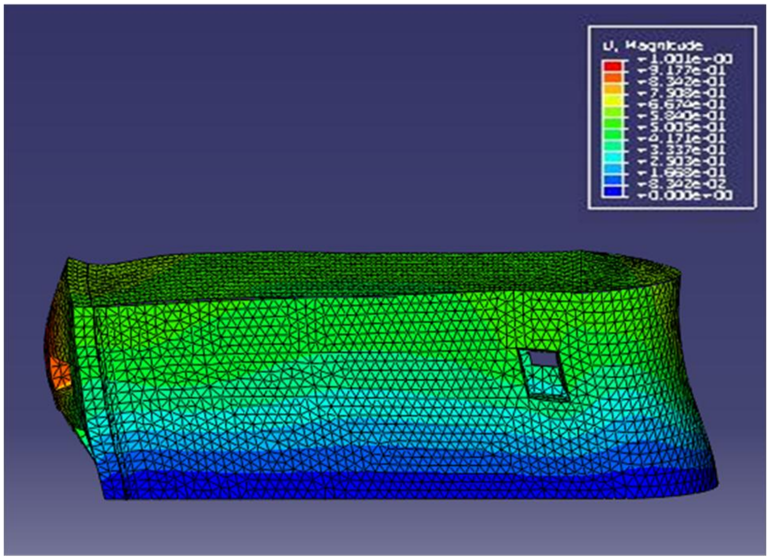
Thickness: 0.60 m
Stone texture: Irregular
Installation: chaotic
Cross section: one
Stone size: 15-25 cm
State of conservation: average
Mortar: poor quality

Fig 17. Masonry rubble stones



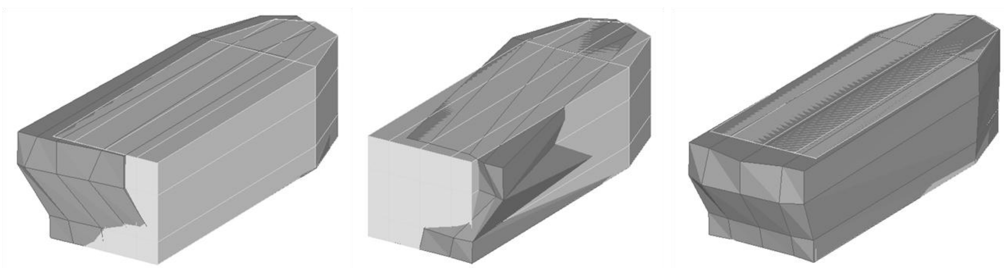
Transverse mode: f= 3.2 Rotational mode : f= 5.30Hz;

Fig 18. Transverse and rotational vibration modes of the *Visitazione* church

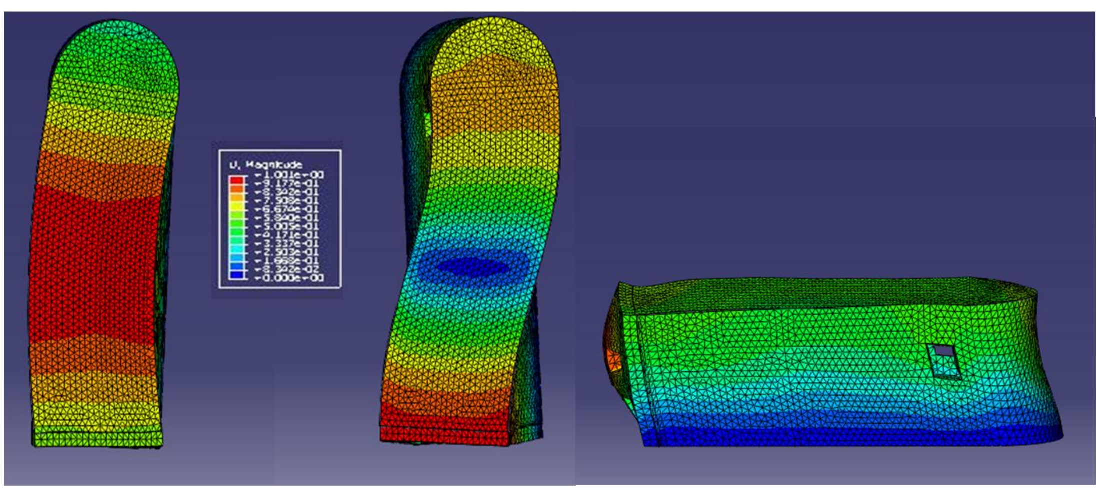


Longitudinal mode : f = 7.35Hz;

Fig 19. Longitudinal vibration mode of the *Visitazione* church



Transverse mode : f = 3.51Hz Rotational mode : f= 5.08Hz; Longitudinal mode : f = 7.32Hz;



Transverse mode: f= 3.2 Rotational mode : f= 5.30Hz Longitudinal mode : f = 7.35Hz

Fige 20. Experimental-numerical comparison in terms of frequencies and modal shapes

Table 1. Dominant frequencies and damping coefficients

Mode Shape	Dominant frequency (Hz)	Damping coefficient (%)
Transverse	3,52	3,3
Rotational	5,08	2,6
-	6,84	2,4
Longitudinal	7,32	1,5
-	11,62	-
-	13,28	-

Table 2. Mechanical properties of masonry rubble stones according to the Italian Code (M. D., 2008)

Masonry typology	f_m **[N/cm^2]**	τ_0 **[N/cm^2]**	E **[N/mm^2]**	G **[N/mm^2]**	w **[kN/m^3]**
Rubble stones	*min–max* 100-180	*min–max* 2-3,2	*min–max* 690-1050	*min–max* 230-350	19

Table 3. Main results of the parametric analyses performed in order to calibrate the FEM model of the church

Mesh size [m]	Mode	E [Nmm^{-2}]						
		345	435	480	525	690	870	1050
		f [Hz]						
0,20	Transverse	3,3692	3,7398	3,9089	4,0693	4,5994	5,1017	5,548
	Rotational	6,0903	6,7395	7,0327	7,3089	8,2079	9,0394	9,762
	Longitudinal	8,4917	9,5055	9,9707	10,4130	11,8830	13,2830	14,53
0,30	Transverse	3,5365	3,9250	4,1022	4,2704	4,8263	5,3531	5,821
	Rotational	6,3277	7,0030	7,3078	7,5950	8,5290	9,3924	10,142
	Longitudinal	8,8746	9,9346	10,4220	10,8850	12,4290	13,9040	15,226
0,40	Transverse	3,6389	4,0383	4,2205	4,3934	4,9651	5,5072	4,220
	Rotational	6,4831	7,1760	7,4887	7,7832	8,7412	9,6265	7,488
	Longitudinal	8,6995	9,7255	10,1950	10,6420	12,1210	13,5280	10,195

Table 4. Mechanical properties of FEM model different elements

Part	Type of ABAQUS Element	Material	Mechanical parameters		
			E [MPa]	*v*	*w* [kN/m^3]
Facade	C3D4	Masonry	345	0,3	19
Lateral wall	C3D4		345	0,3	19
Apse	C3D4		345	0,3	19
Arch	C3D4		345	0,3	19
Device	C3D4		1050	0,3	19
Corner	C3D4		1	0,5	19
Truss	C3D4	Wood	9000	0,35	4,50
Planking	C3D4		1000	0,35	4,50

Table 5. Comparison among experimental and numerical frequencies

Mode	Experimental frequency [Hz]	Numerical frequency [Hz]
Transverse	3.52	3.20
Rotational	5.08	5.30
Longitudinal	7.32	7.35