Ambient vibration tests on a monumental palace in Castelnuovo of S.Pio (AQ)

Antonio Formisano
Dipartimento di Ingegneria Strutturale – Università di Napoli "Federico II". P.le V. Tecchio 80, 80125 Napoli

Gilda Florio, Raffaele Landolfo
Dipartimento di Costruzioni e Metodi Matematici in Architettura – Università di Napoli "Federico II". Via Forno Vecchio 36, 80134 Napoli

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ABSTRACT
The experimental activity presented in this paper is related to in-situ vibration tests performed on a monumental masonry building placed in Castelnuovo of San Pio (AQ). The purpose of this experimental measurement campaign was the achievement of the dynamic characteristics of the palace, which have been expressed in terms of both natural frequencies and vibration shapes. The tests was carried out in May 2010 within the scientific cooperation between the University of Naples "Federico II" and the University St. "Cyril and Methodius" of Skopje established in the framework of the COST C26 Action project "Urban Habitat Constructions under Catastrophic Events".
The achieved experimental results have been numerically reproduced on the basis of a FEM model of the building implemented within the ABAQUS analysis software. Also, a numerical model of the palace before earthquake has been set-up. The behavioural difference in terms of vibration periods between two numerical models has allowed to estimate the damages occurred into the palace after earthquake. Finally, a linear modal analysis has been also performed on a retrofitted FEM model of the building. The output of the analysis represents a clear demonstration of the effectiveness of the proposed design intervention.

1 L'AQUILA EARTHQUAKE

On April 6^{th}, 2009 at 3:32 a.m. (1.32 UTC) an earthquake stroke the Abruzzo region, a 5000 km^2 area located within the Central Apennines of Italy. In particular, the capital of the region, namely L'Aquila, a city of about 73.000 people, and several villages of the middle Aterno valley were mostly hit by the seism.

The mainshock was rated 5.8 on the Richter Scale (M_L) and 6.3 on the Moment Magnitude Scale (M_W). Although the epicentre depth was not so deep, the seismic waves associated with shallow quakes produced very strong shaking and many damages; therefore, the mainshock was followed by many aftershocks (Fig.1).

The earthquake was generated by a normal fault, located in a valley contained between two parallel mountain along the direction North-South (Fanale et al., 2009), with a maximum vertical dislocation of 25 cm and hypocentre depth of about 8.8 Km (Fig. 2).

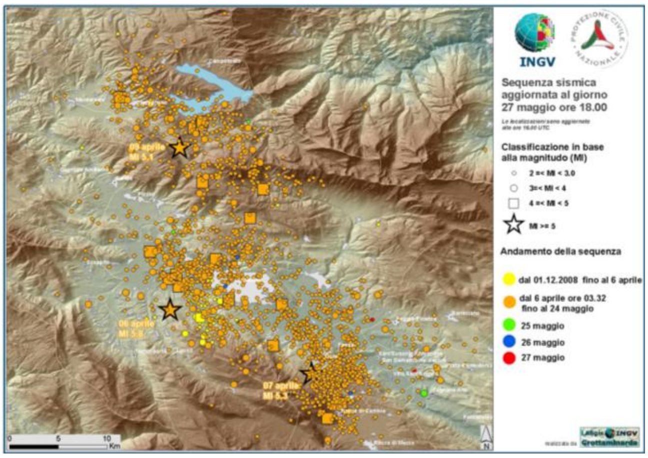


Figure 1. Seismic sequence and location of the L'Aquila earthquake epicenter (INGV)

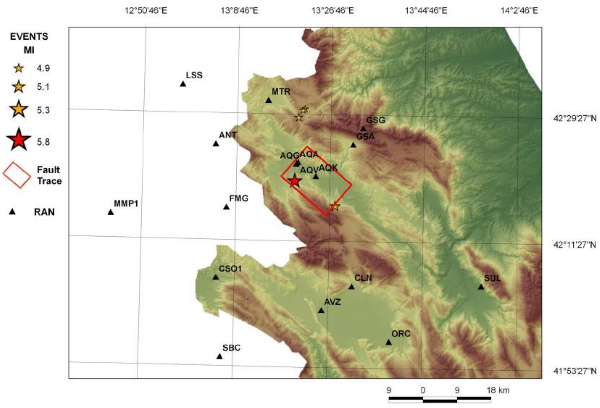


Figure 2. Fault geometry

It was the third main earthquake recorded in Italy since 1972, after the Friuli event (1976; M_w=6.4) and the Irpinia one (1980; M_w=6.9). Furthermore, this seismic event was the strongest one among a sequence of 23 quakes having M_w>4 and occurred between 2009 March, 30th and 2009 April, 23rd (Fig. 3), it providing strong motion recordings from accelerometer stations placed very close (4-5 Km) to the epicentre (Fig. 4).

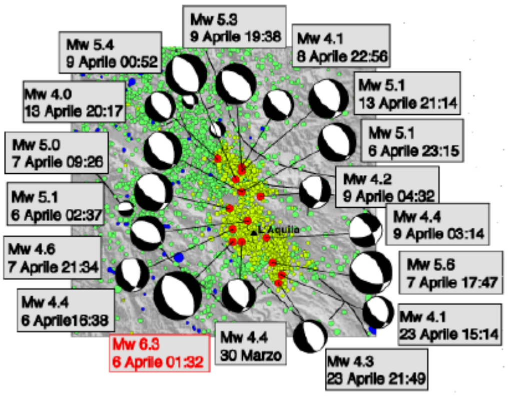


Figure 3. Sequence of seismic events occurred in the L'Aquila district (INGV)

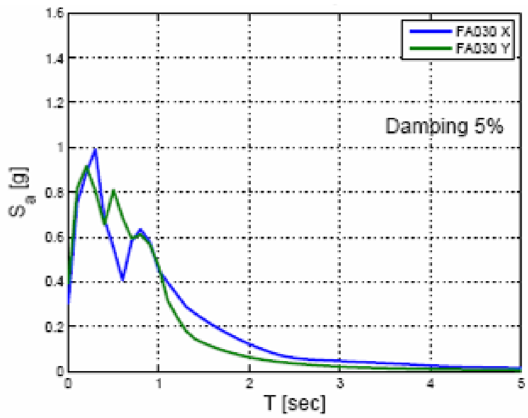


Figure 4. Elastic acceleration spectrum 4.3 km far from the epicentre

L'Aquila earthquake is considered as an exceptional seismic action, since the maximum recorded horizontal and vertical acceleration components within the epicentral area were larger than PGAs of the elastic spectra given by the Italian Code (M. D., 2008) (Table 1). Also, this event represents the first well-documented strong motion earthquake instrumentally recorded in Italy in a near-fault area (Monaco et al., 2010).

The main event was recorded by 56 digital strong motion stations, which are part of the Italian Strong Motion Network (in Italian *Rete Accelerometrica Nazionale*, RAN), owned and maintained by the Department of Civil Protection (DPC). The PGA map for the M_L 5.8 mainshock on 1:32 GMT, April 6th is depicted in Figure 5.

Table 1.Earthquake effects at different epicentre distances

Record identifier	Dir. x	Dir. y PGA (g)	Dir. z	Epicentre distance (Km)
GX066	0.626	0.597	0.420	4.8
FA030	0.416	0.434	0.215	4.3
CU104	0.394	0.451	0.380	5.8
AM043	0.342	0.340	0.350	5.6
EF021	0.153	0.149	0.112	18.0
TK003	0.081	0.089	0.045	31.6

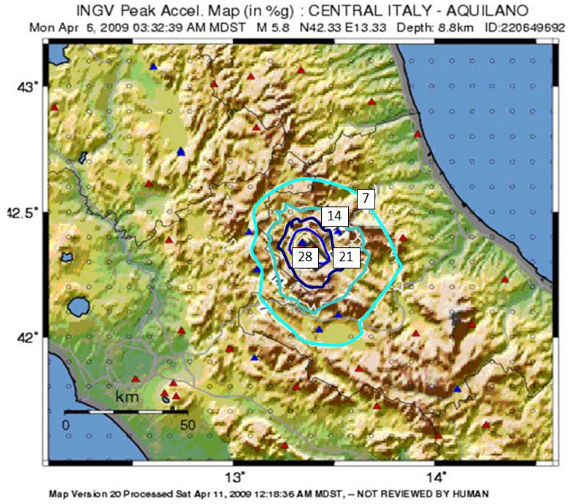


Figure 5. Peak Ground Acceleration (PGA) map for the M_L 5.8 mainshock (INGV)

The distribution of the damages within the affected area was non-uniform and asymmetric.

The mainshock caused heavy damages in the centre of L'Aquila, where intensity value was reported varying between VIII and IX. Damages were even more significant in some villages located in the middle Aterno valley, where intensities as high as IX-X were experienced in Castelnuovo and Onna (Figs. 6 and 7). In total, 14 municipalities experienced a MCS intensity between VIII and IX, whereas those characterized by MCS intensity *I* larger than VII were altogether 45 (Galli and Camassi, 2009).

After the earthquake about 10.000-15.000 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 €.

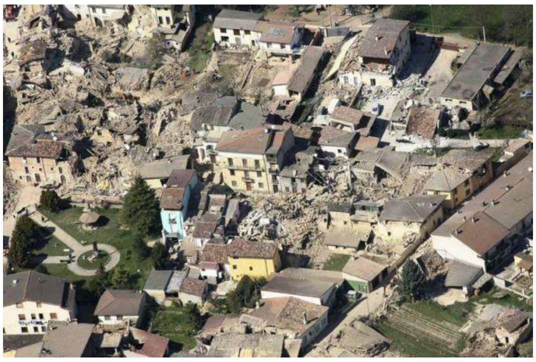


Figure 6. The historical centre of Onna after L'Aquila earthquake

Figure 7. The historical centre of Castenuovo of San Pio after L'Aquila earthquake

2 THE INVESTIGATED BUILDING

Castelnuovo of San Pio is a hamlet of the municipality of San Pio delle Camere. The ancient walled nucleus is situated on the top of a hill, while an irregular urban area following the contours develops on the mountainside. The urban scheme of the town high part is regular and develops according to the so-called chessboard or hippodamian plan, in which all the streets are orthogonal to each other. The whole rectangular area identified by these streets has dimensions of 70 m x 56 m and it is divided into four blocks. Formerly, the entrance of the ancient village was a round arch and, probably, the walled zone was surrounded by a moat (http://www.castelnuovoonlus.com/castelnuovo).

Before the earthquake, the historical centre mostly consisted of 2-3 stories ordinary buildings characterized by a low quality masonry structure.

The seismic event produced many damages and several collapses. However, it is worth to be noted that a lower damage level has been observed on the buildings at the toe of the hill as respect to the constructions located on the hilltop. This aspect is due to some factors related to topographic amplification, which have contributed to the strong shaking at the highest elevation of the village (Monaco et al., 2009).

On the hilltop of the village the most important monuments, that is the St. Giovanni Battista Church, built in 1703 on the ruins of another chapel, and the Sidoni Palace, are located.

The Sidoni Palace (bird's-eye view in Fig. 8) is a monumental building in the old medieval nucleus of Castelnuovo. It is an isolated building, having a regular and symmetric plan shape and developing on two floors (Fig. 9). The façade is also symmetric and characterized by regular openings and several architectural ornaments (Fig. 10). The entrance is constituted by an arch located in the central part of the façade.

Figure 8. Bird's-eye view of the Sidoni Palace

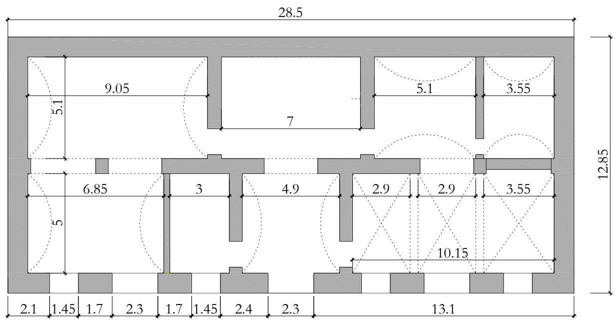


Figure 9. Plan view of the Sidoni Palace

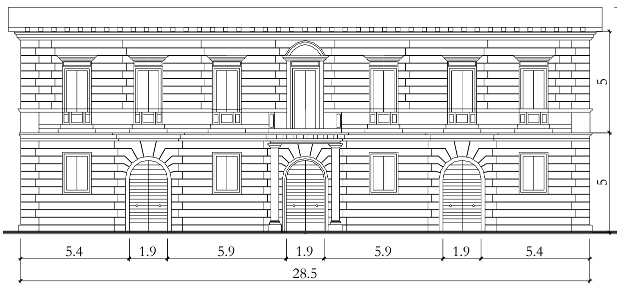


Figure 10. Front view of the Sidoni Palace

The building is a sack stone masonry structure with a ground floor developed on a rectangular surface of about 366 m^2 and covered by tunnel vaults. The first level floor is realised with steel profiles and hollow flat tiles, whereas a wooden pitched roof, covered by clay tiles and rebuilt after the demolition of the original vaulted roof, represents the building coverage.

The building has a basement floor and also the ground storey is on one side at contact with the ground, as it shown in the transversal section of the palace depicted in Figure 11.

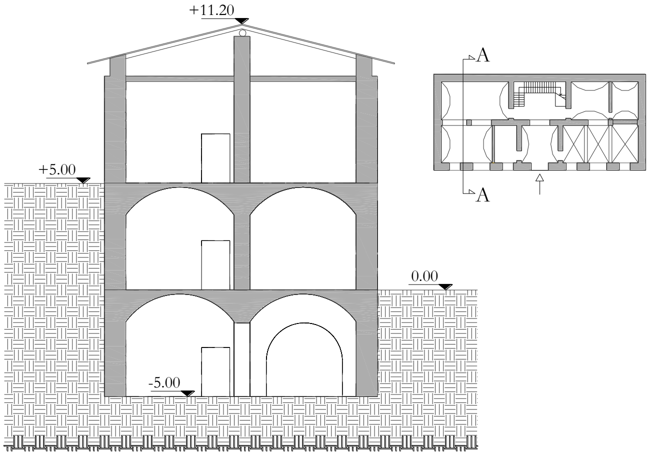


Figure 11. Transversal section of the Sidoni Palace

After earthquake no damage was recorded in the building facade, whereas significant damages and collapses of part of masonry vaults took place inside (Fig. 12). Moreover, some of the steel floors at the first level of the building were destroyed by the earthquake, as illustrated in the building 3D views plotted in Figure 13.

Figure 12. Damages occurred in the vaulted ceilings

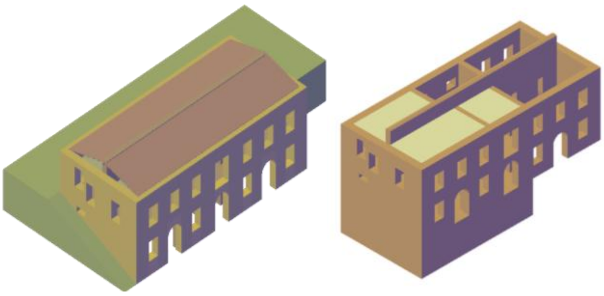


Figure 13. 3D views of the building

3 THE EXPERIMENTAL AMBIENT VIBRATION TESTS (AVT)

3.1 *General issues*

The ambient vibration testing method is a widely applied and popular full-scale testing method for experimental definition of structural dynamic characteristics of a construction. It is based on measuring the structural vibrations caused by ambient vibration sources (Fig. 14). In particular, as input signals the wind, the traffic noise or some other micro-tremor and impulsive forces, like wave loading or periodical rotational forces of some automatic machines, can be considered.

The ambient vibration test is a non-destructive test which represents a special investigation method purposely conceived especially for dynamic identification of buildings having a historical and artistic importance. Moreover, the method is very fast and the relatively simple implementation procedure can be performed on a structure in use, without disturbing its normal functioning.

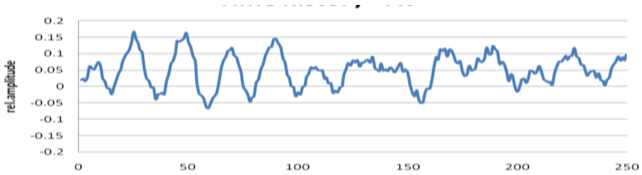


Figure 14. Time history of vibrations excited by the ambient

The basic assumption of this method is that the excitation forces are a stationary random process, having an acceptably flat frequency spectrum. In such conditions, the structures will vibrate and their response will contain all their normal modes (Krstevska et al., 2008).

The ambient vibration testing procedure consists of real time recording of the vibrations and processing of the records. The initial test is the dynamic calibration test. During this test all sensors, namely the seismometers, are placed on the same position and in the same direction. So, the signals are recorded simultaneously and the related Fourier spectra are obtained. Resonant frequencies of the structure can be preliminary defined using the dynamic calibration tests, but the final definition of the natural frequencies is possible after obtaining the mode shapes of vibration. After this calibration test, the seismometers are placed at both different levels and different points of the structure, but along the same direction, for simultaneous recording. This is necessary for obtaining the mode shapes of vibration. Since the input – output correlation is not a priori known, a steady point must be fixed as a Reference Point (RP), it being usually chosen at the highest level of the structure. In this way each measured point is normalised as respect to the RP amplification and the global dynamic response is identified. The duration of the recording should be long enough to eliminate the influence of possible non-stochastic excitations which may occur during the test.

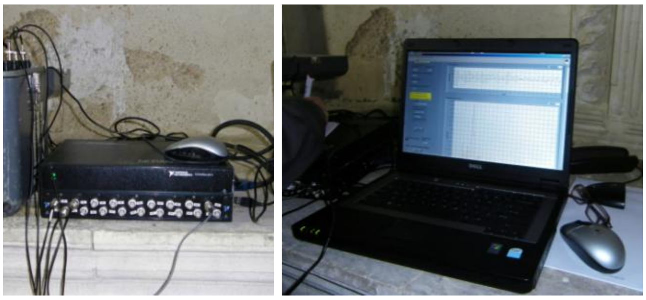
3.2 *Testing equipment*

During the ambient vibration measurements of Sidoni Palace, three seismometers Ranger type Kinematrics product were used (Fig. 15) and the measured signal was amplified by four channel signal conditioner (Fig. 16). The amplified and filtered signals from the seismometers were then collected by high-speed data acquisition system (Fig. 17a), which transformed the analogue signals into digital ones. PC (Fig. 17b) and special software for on-line data processing have been used to plot the time histories of recorded velocities by the seismometers together with the Fourier Amplitude Spectra (FAS) of the response at any recorded point.

Figure 15. Equipment for ambient vibration measurements: the Ranger type seismometers

a) *b)*

Figure 16. Equipment for ambient vibration measurements: Kinemetrics product *(a)*; four channel signal conditioner *(b)*



a) *b)*

Figure 17. Equipment for ambient vibration measurements : high-speed data acquisition system *(a);* computer for data processing *(b)*

3.3 *Post-processing and experimental results*

Measurements of the ambient vibrations on Sidoni Palace in Castelnuovo were performed along transversal and longitudinal directions of the building at selected structure points depicted in Fig. 18 on the geometrical model of the structure generated by the ARTeMIS software (Krstevska et al., 2010). The total number of the performed tests was 38, they including also the dynamic calibration tests. The data sets consisted on records of velocity signals with duration of 100 seconds and the sampling frequency was 200 S_a/s.

For post-processing and analysis of the recorded vibrations in all measuring points, the ARTeMIS extractor software was used. This software is based on both the Frequency Domain decomposition and the Peak Picking technique, it allowing for a good graphical presentation of the obtained results.

The Peak Picking of the dominant frequencies obtained by means of the ARTeMIS software is plotted in Figure 19.

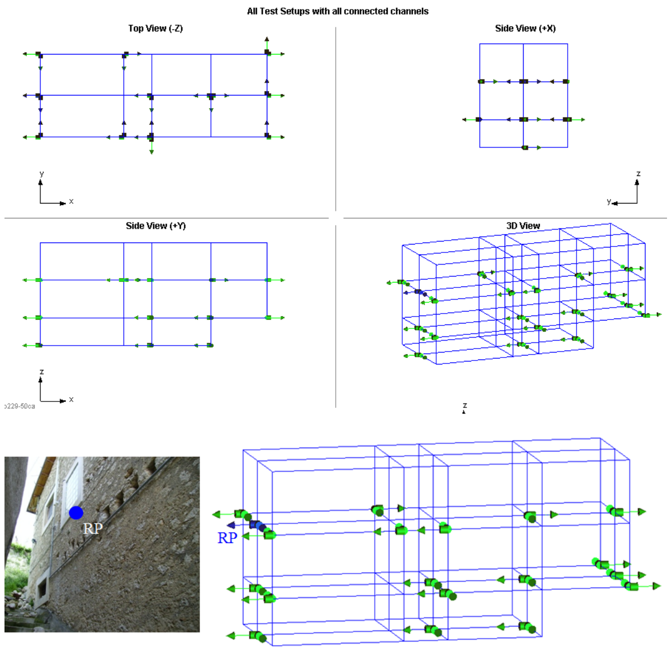


Figure 18. Top, side and 3D views of the geometry of the palace with positioning of the measuring points (RP is the blue point).

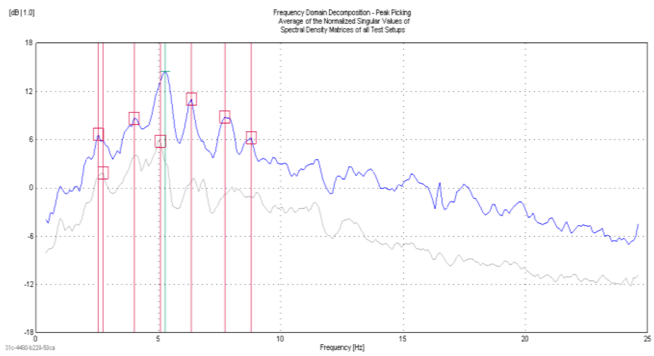


Figure 19. Peak Picking of the dominant frequencies of the Sidoni Palace

From this figure it is apparent that the spectrum is characterised by several dominant frequencies. The vibration shapes at particular frequencies are given from Fig. 20 to Fig. 22. It is evident that the most clear frequency *f* for transversal vibration is at 5.08 Hz, while for longitudinal and rotational modes the frequencies of 5.27 Hz and 6.35 Hz are respectively noticed.

Measured vibrations have been subsequently analysed in frequency domain up to 25 Hz. The values of the dominant frequencies are specified in Table 2, along with the corresponding damping coefficients ranging from 2.1% to 5.6 %.

Table 2. Dominant frequencies and damping coefficients obtained on the Sidoni Palace

Dominant frequency (Hz)	Damping coefficient (%)
2.54	5.6
2.73	-
4.0	3.0
5.08	4.5
5.27	5.2
6.35	3.8
7.7	2.1
8.8	2.6

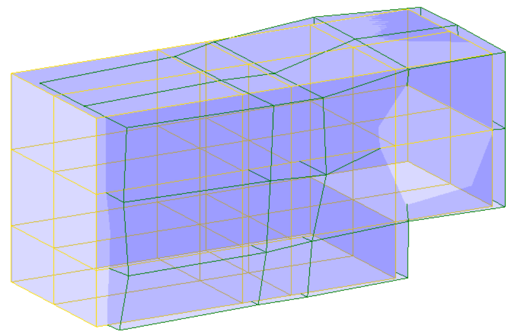


Figure 20. Vibration shape at a frequency of 5.08 Hz (transverse mode)

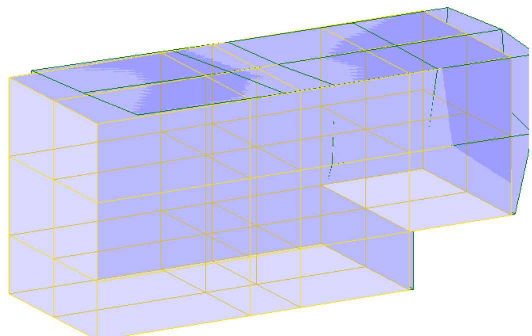


Figure 21. Vibration shape at a frequency of 5.27 Hz (longitudinal mode)

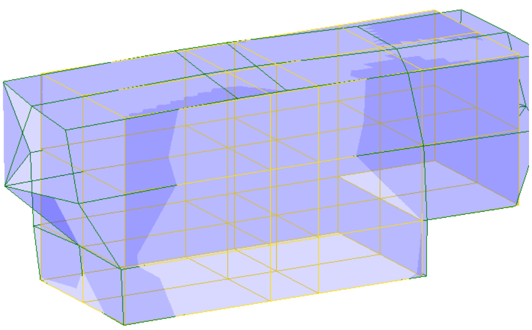


Figure 22. Vibration shape at a frequency of 6.35 Hz (rotational mode)

4 NUMERICAL ACTIVITY

4.1 Calibration of experimental results

The dynamic response of the Sidoni Palace has been investigated by means of numerical frequency analyses implemented within the ABAQUS computer code environment after the three-dimensional solid model of the structure created in a computer aided design system has been imported (Florio, 2010).

In order to properly assess the structural interaction among the different constituent parts of the building, the geometrical model accurately reproduces all its main components, including openings, vaults and horizontal floors. With regard to this last aspect, it is worth to precise that the collapsed floors have not been included in the model, in order to take into account the real damage state of the building.

The whole masonry structure, fully restrained at its base, has been discretized by means of tetrahedral 3D solid elements, namely C3D4 (4-node linear tetrahedron) elements, having size length of 0.40 m (Fig. 23) (HKS, 2004).

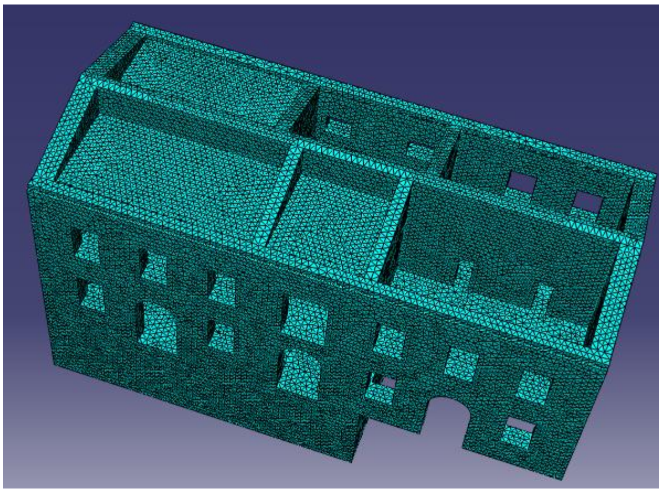


Figure 23. The ABAQUS FEM model of the palace

As far as the material modelling is concerned, since the analysis is aimed at the structural global response identification, a continuum homogeneous material has been assumed. In addition, considering that the frequency analysis is purely a linear perturbation analysis type, only linear elastic properties of the masonry are required. So, the density has been assumed on the basis of the provisions of the Italian Ministerial Circular (M. C., 2009), it being equal to 19 kN/m^3, while the elastic modulus *E* has been opportunely calibrated on the basis of flat jack tests performed on masonry (Fig. 24).

The experimental in-situ test has provided the stress- vertical (Fig. 25) and horizontal (fig. 26) strain laws.

Flat jack test parameters

A_C, *Cutting area*	882 cm^2
A_J, *Flat jack area*	778 cm^2
Jacks distance	30 cm
K_m , *Jack constant*	0.78
K_a [A_J/A_C]	0.88

Figure 24. Flat jack test on masonry: test set-up

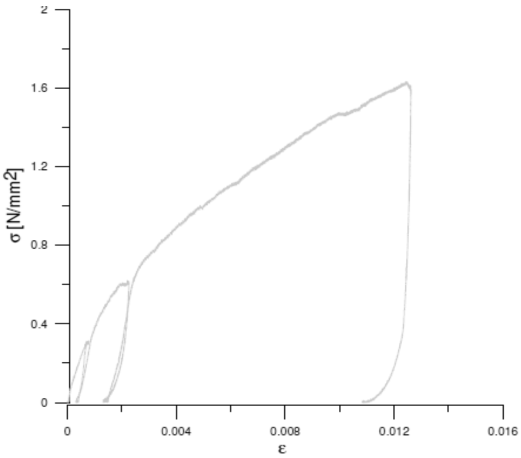


Figure 25. Stress vs. vertical strain law of masonry

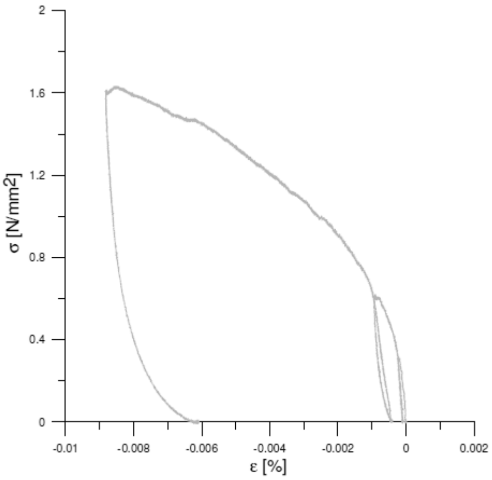


Figure 26. Stress vs. horizontal strain law of masonry

Considering also the indications provided in literature, a Young modulus E ranging between $500.f_k$ and $1000.f_k$, where f_k is the compressive strength of masonry, has been considered. So, the elastic modulus has been assumed to be variable within the range [800-1600] MPa.

First, several analyses have been carried out by changing the value of E in order to find both the frequencies and the mode shapes that better approximated the experimental ones (f_{tran}=5.08 Hz; f_{long}= 5.27 Hz). The achieved results are shown in Table 3, where it is apparent that the best results are obtained by adopting an elastic modulus E=1200 MPa. Such a value is not within the elastic modulus range provided by the Italian M.C. (2009) for the considered type of masonry.

Table 3. Numerical dominant frequencies corresponding to different E values

E (MPa)	Mode frequency *f* (Hz) Transverse	Longitudinal
800	4.33	5.10
1000	4.80	5.07
1200	5.26	5.30
1300	5.47	5.52
1600	6.07	6.12

Afterwards, a mesh sensitivity analysis has been carried out in order to refine the obtained results. Thus, by adopting the criterion that each finite element contained at least two stones or little more than two stones along the length, the following four mesh sizes have been considered:

1. *Coarse Mesh*: side length 0.50m;
2. *Medium Mesh*: side length 0.40m;
3. *Fine Mesh*: side length 0.30m;
4. *Very fine Mesh:* side length 0.20m.

Later on, a frequency analysis has been performed individually for each of the FEM models characterised by the four above meshes and the corresponding natural frequencies have been found for the first 10 vibration modes. In particular, it has been noticed that the analysis on the numerical model with *very fine* mesh has been not concluded since computational efforts are too high in comparison to the computer capability.

Conclusively, by comparing the analysis time with the accuracy of results, it has been found that the *fine mesh* is able to better approximate the experimental results in terms of both deformed shape and frequency, especially with reference to the first transverse mode (Table 4).

Table 4. Dominant frequencies and damping coefficients obtained on the Sidoni Palace FEM model

Mesh type	Transverse *f* [Hz]	Longitudinal *f* [Hz]
Coarse	5.36	5.42
Medium	5.26	5.30
Fine	5.06	5.17
Very fine	Analysis not concluded	

The deformed FEM model configurations corresponding to the fundamental modal shapes are depicted in Figures 27, 28 and 29. From these figures, it may be pointed out that, according to the experimental results, the most deformable parts of the palace are masonry walls adjacent to the collapsed floors.

In Table 5 the experimental natural frequencies are compared with the ones achieved under numerical way by using the FEM model with fine mesh. It is worth to precise that the first global transverse vibration mode numerically founded had a frequency equal to 3.92 Hz. From the comparison, a reasonable agreement of results both in terms of vibration modes and natural frequency values is noticed (see Figs. 27-29).

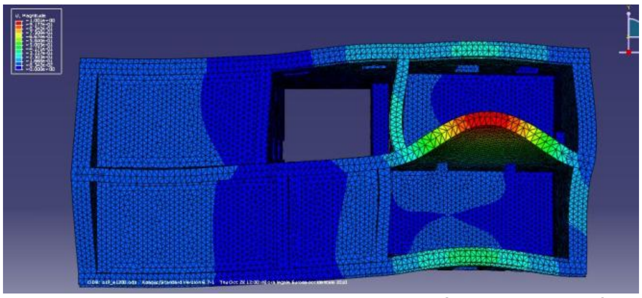


Figure 27. Transversal vibration mode (f = 5.06 Hz) of the actual building FEM model.

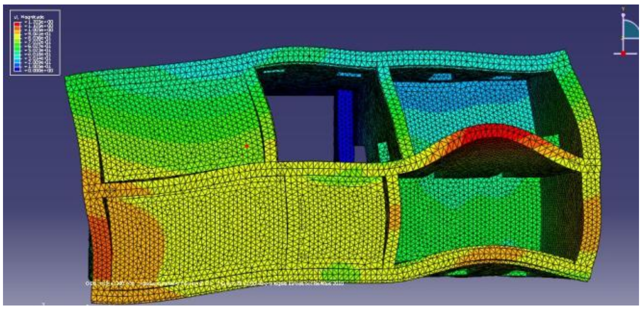


Figure 28. Longitudinal vibration mode (f = 5.17 Hz) of the actual building FEM model.

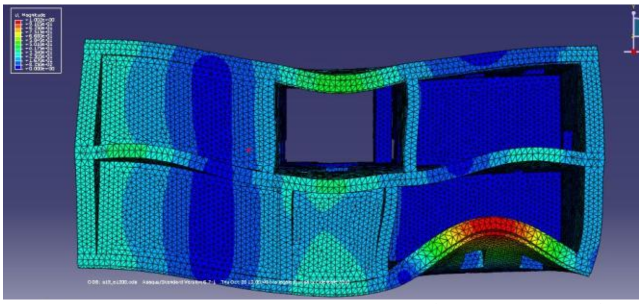


Figure 29. Torsional vibration mode (f = 6.59 Hz) of the actual building FEM model.

Table 5. Comparison between experimental and numerical frequencies

Experimental frequency Hz]	Numerical frequency [Hz]
4.10	3.92
5.08	5.06
5.27	5.17
6.35	6.59
7.70	7.21

4.2 *Modal behaviour of the original building*

The frequency analysis has been also performed on the FEM model representative of the structural configuration of Sidoni Palace before the seismic event of 2009, April 6th. The essential difference between the original building and the actual one is that in the former some more floors were present, they being collapsed under earthquake.

The global modal vibration shapes of the palace are illustrated in Figures 30, 31 and 32. It may be noticed that in this undamaged configuration the global mode shapes are clearly defined in each directions. In particular, the large transverse deformation of masonry walls due to the lack of floors in the damaged building is missed.

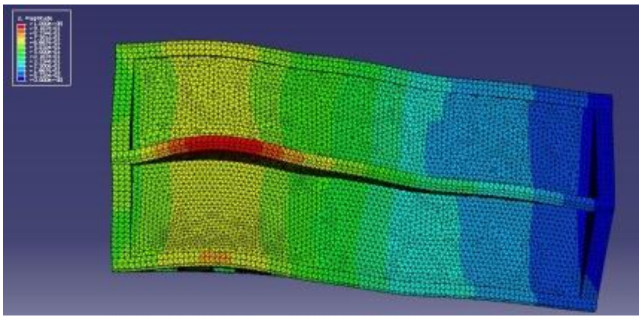


Figure 30. Transversal vibration mode (f = 4.00 Hz) of the building FEM model before earthquake.

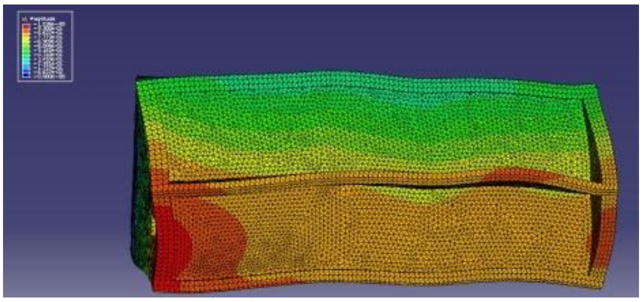


Figure 31. Longitudinal vibration mode (f = 5.32 Hz) of the building FEM model before earthquake.

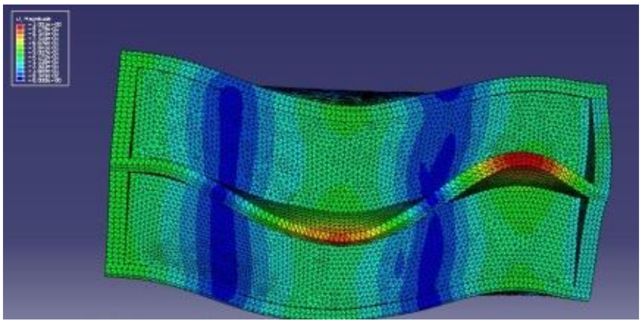


Figure 32. Torsional vibration mode (f = 7.09 Hz) of the building FEM model before earthquake.

A difference in terms of stiffness can be noticed between the original building and the damaged one (Table 6). In particular, in order to make a comparison between homogeneous modal shapes, the first transverse vibration mode of the undamaged configuration (corresponding to a frequency of 4.00 Hz) has been compared with the first global one of the damaged configuration (frequency equal to 3.92 Hz).

The comparison reveals that the stiffness of the damaged model averagely decreases of the order of 4% with respect to the one of the building before earthquake.

Table 6. Comparison among frequencies

Damaged model f [Hz]	Undamaged model f [Hz]	Stiffness decrease [%]
3.92	4.00	2
5.17	5.32	3
6.59	7.09	7

4.3 Retrofitting intervention

The recent seismic event has evidenced the high structural vulnerability of Sidoni Palace, mostly due to low effectiveness of both connections among walls and floor-to-wall connections. Thus, on the basis of the achieved results, retrofitting measures have been proposed for the restoration of the examined construction

At first, the rebuilding of the collapsed floors and the replacement of the existing ones with mixed RC beams-hollow tiles ones have been proposed. In fact, RC floors behave as rigid diaphragms, assuring an adequate distribution of the seismic forces to all of the bearing walls. Therefore, these floors have to be well connected to the walls by means of the arrangement of tie beams.

Second, the repairing of damaged vaults and the placement of metal ties have been planned. The use of metal ties, indeed, represents an ancient and widespread intervention technique used to eliminate the horizontal thrust of arches, vaults and roofs. This system is an effective and reliable technique to obtain a better connection between structural elements at the floor level, ensuring a box-type behaviour of the entire structure. Moreover, this technique allows to avoid the possible out-of-plane overturning mechanisms of masonry walls.

The tie-bars intervention technique herein proposed consists of the insertion of metal bars in the wall along the vault length in both the two orthogonal directions.

Furthermore, other possible interventions have been considered: the replacement of fractured brackets; the relocation of the fallen stones and joints filling; the consolidation of the wooden roof by means of the arrangements of perimeter tie beams.

Taking into account all the aforesaid interventions, a retrofitted FEM model has been implemented by means of the ABAQUS code. In the specific case, the constraint conditions among the new RC floors and the bearing walls have been improved in order to consider the active presence of the designed tie beams. Moreover, beam elements have been inserted in the wall in order to model the metal ties.

Afterwards, frequency analyses have been achieved on this retrofitted model, providing the modal shape depicted in Figures from 33 to 35.

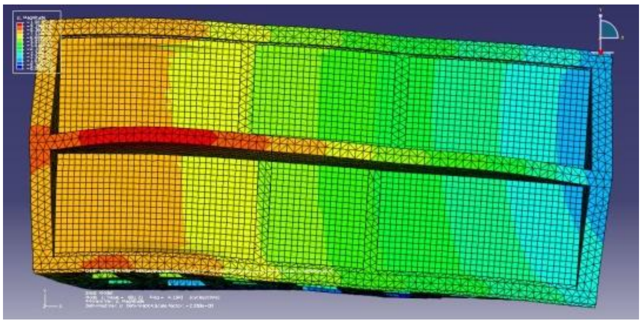
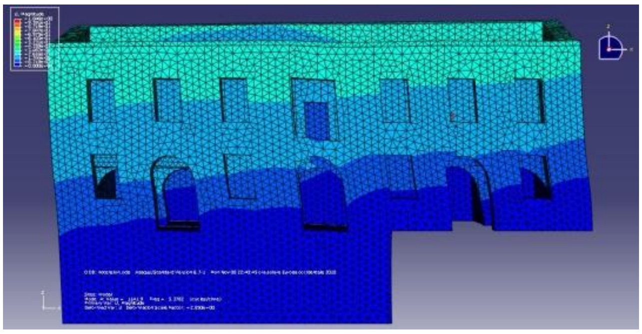


Figure 33. Transverse vibration mode (f = 4.15 Hz) of the retrofitted building FEM model.



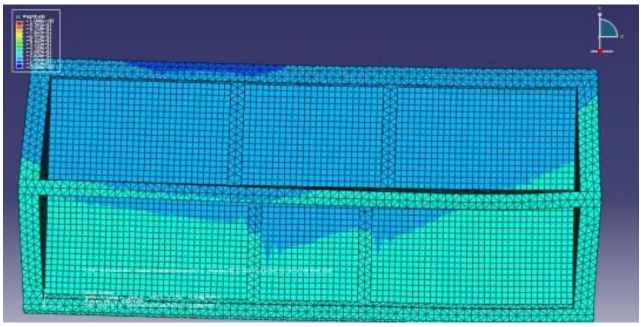


Figure 34. Longitudinal vibration mode (f = 5.37 Hz) of the retrofitted building FEM model.

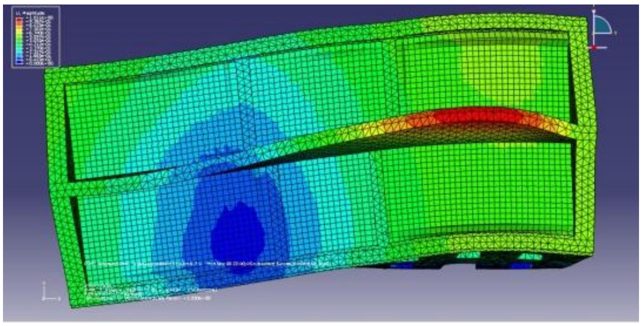


Figure 35. Torsional vibration mode (f = 6.41 Hz) of the retrofitted building FEM model.

By viewing the previous figures and also the results provided in Table 7, it is noted that the modal behaviour of the structure is drastically changed, since modal shapes are clearly defined, and the local modes coincide with the superior ones. In addition, the structural stiffness increases of the 6% for the transverse and longitudinal fundamental vibration modes. Definitively, the modal behaviour of the building is totally improved.

Table 7. Comparison among the dominant frequencies of the damaged model and the retrofitted one

Damaged *f* [Hz]	Retrofitted *f* [Hz]	Stiffness increase [%]
3.92	4.15	6
5.06	-	-
5.17	5.37	6
-	6.41	-
6.59	9.21	28

5 CONCLUSIONS

The ambient vibration testing method herein applied on the Sidoni Palace in Castelnuovo of San Pio was performed with the objective to obtain the dynamic characteristics of the damaged structure after the 2009 L'Aquila earthquake.

The test results have shown that in the domain from 0 to 25 Hz several frequencies are dominating. In particular, the most clear modes in transversal and longitudinal direction have frequencies of 5.09 Hz and 5.27 Hz.

The tests have also provided the values of the equivalent damping coefficients, they ranging from 2.1% to 5.6%.

Then, the dynamic characteristics of the palace obtained from experimental testing have been also achieved under numerical way after a FEM model of the building by means of the ABAQUS non linear numerical code has been implemented. In this numerical activity the elastic features of masonry have been determined from the experimental flat jack tests performed on the building masonry walls.

The obtained results have shown a good agreement between experimental natural frequencies and mode vibration shapes. Thus, in this phase, the numerical simulation of experimental tests has been of fundamental importance for detecting the building behaviour under a future possible earthquake. Therefore, appropriate temporary safeguard measures could be implemented in order to make safe the building at the actual state.

Frequency analysis has been also carried out considering an undamaged building model based on the original structural configuration. This investigation has permitted to evaluate the reduction in stiffness of the damaged palace with reference to the original one after earthquake.

Later on, appropriate retrofitting interventions have been proposed, their effectiveness being proved by performing numerical analyses on the improved FEM model of the palace. Thanks to these interventions, the modal behaviour of the retrofitted structure has been drastically improved, with clearly defined modal shapes and a considerable increase of the frequencies related to each fundamental vibration mode with respect to the ones deriving from the damaged building.

As further development, it is recommended to repeat the AVT measurements after strengthening measures are set-up in order to both check the effects of the applied strengthening solution and assess the dynamic properties of the new structure.

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REFERENCES

Fanale L., Lepidi M., Gattulli V., Potenza F., 2009. Analysis of buildings damaged from the 2009 April seismic event in the town of L'Aquila and in some neighbouring minor centres (in Italian). *DISAT publication n. 03.*

Florio G., 2010. *Vulnerability of historical masonry buildings under exceptional actions.* Ph.D Thesis, University of Naples "Federico II".

Galli P., Camassi R., 2009. Report on L'Aquila earthquake of 6th April 2009 effects (in Italian). *Joint Report DPC-INGV.*

http://www.castelnuovoonlus.com/castelnuovo.

HKS, 2004. *ABAQUS Theory Manual*. USA.

Krstevska L., Kustura M., Tashkov L., 2008. Experimental in-situ testing of reconsructed old bridge in mostar. *Proc. of the 14th World Conference on Earthquake Engineering*, Beijing, China, October 12-17.

Krstevska L., Tashkov L., Naumovski N., Florio G., Formisano A., Fornaro A., Landolfo R., 2010. In-situ experimental testing of four historical buildings damaged during the 2009 L'Aquila earthquake. *Proc. of the COST C26 Final International Conference*, Naples, September 16-18.

Ministerial Circular (M. C., 02/02/2009). *Instructions for the application of the "New Technical Code for Constructions" (M. D: 14/01/2008).* Official Gazette of the Italian Republic published on February 26th.

Ministerial Decree (M. D., 08), 2008. *Technical codes for constructions*. Official Gazette of the Italian Republic published on January 14th.

Monaco P., Totani G., Barla G., Cavallaro A., Costanzo A., D'Onofrio A., Evangelista L., Foti S., Grasso S., Lanzo G., Madiai C., Maraschini M., Marchetti S., Maugeri M., Pagliaroli A., Pallara O., Penna A., Saccenti A., Santucci de Magistris F., Scasserra G., Silvestri F., Simonelli A. L., Simoni G., Tommasi P., Vannucchi G., Verrucci L., 2009. Geotechnical aspects of the L'Aquila earthquake. *Proc. of the XVII International Conference on Soil Mechanics & Geotechnical Engineering*, Alexandria, Egypt.