Experimental ambient vibration tests and numerical investigation on the Sidoni Palace in Castelnuovo of San Pio (L'Aquila, Italy)

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Abstract: In the present paper the experimental and numerical activities related to in-situ Ambient Vibration Tests performed on a monumental masonry building placed in Castelnuovo of San Pio (Italy) are presented. The obtained experimental results, able to identify the dynamic characteristics of the investigated palace, have been successively reproduced by means of a building FEM model through the ABAQUS calculation code. Numerical frequency analysis has been also performed on the original building aiming at evaluating its reduction in stiffness due to earthquake. Finally, appropriate retrofitting interventions have been proposed, their effectiveness being proved by numerical analyses on the improved palace FEM model.

Keywords: Abruzzo earthquake; ambient vibration test; AVT; monumental masonry building; FEM model; vulnerability curves; retrofitting interventions.

Reference to this paper should be made as follows: Formisano, A., Krstevska, L., Di Lorenzo, G., Landolfo, R. and Tashkov, L. (2018) 'Experimental ambient vibration tests and numerical investigation on the Sidoni Palace in Castelnuovo of San Pio (L'Aquila, Italy)', *Int. J. Masonry Research and Innovation*, Vol. 3, No. 3, pp.269–294.

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This paper is a revised and expanded version of a paper entitled 'In-situ experimental testing of four historical buildings damaged during the 2009 L'Aquila earthquake' presented at the *Final Conference of the COST ACTION C26 project "Urban Habitat Constructions under Catastrophic Events"*, Naples, Italy, 16–18 September, 2010.

1 Introduction

Environmental hazards (floods, landslides, volcano eruptions, earthquakes, etc.) are causes of major disasters in European countries. Among these exposures, earthquake was the most important cause of destruction, it affecting especially cultural heritage.

Within this framework, two categories of architectures, namely monumental buildings and historical ones, can be distinguished.

Monumental buildings are existing constructions having important cultural values, that is historical meaning and typological and material aspects within a general architectural interest, as well as a valuable testimony of ancient times, which deserve to be preserved. Historical buildings are constructions having themselves a cultural value as a whole, since they are placed in a historical urban area, while they considered as isolated structures cannot be considered as a monument. Buildings placed in historical areas, and thus named historical constructions, require a systematic study of that whole urban region from different viewpoints, namely historical, architectural, technological and seismic (micro-zoning), in order to both evaluate their seismic safety and indicate possible measures for improving their behaviour against earthquakes. Therefore, interventions on historicals building may be more incisive in some aspects than those regarding monumental buildings, if the general urban character is conserved.

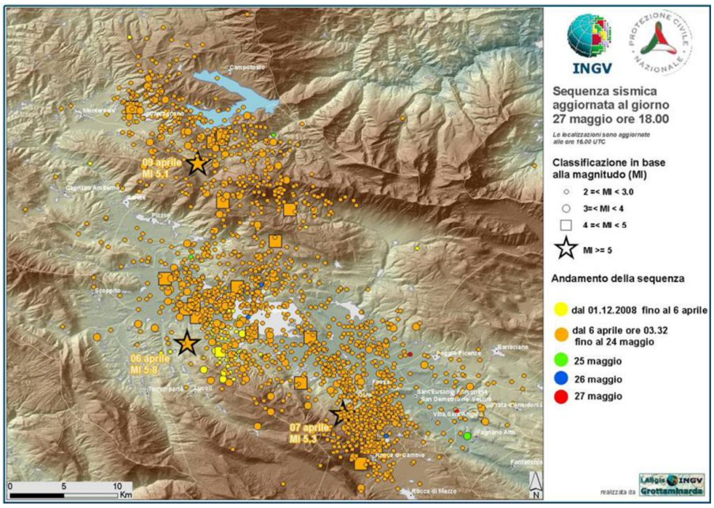
Actually in Italy for seismic vulnerability assessment of monumental buildings, specific Guidelines on Cultural Heritage (Italian Minister for Architectonic and Cultural Heritage (MiBAC), 2011) can be used. According to these guidelines, three different analysis approaches are employed: LV1 method, used for large scale seismic evaluation of historic built-up, LV2 method, applied for local mechanisms analysis of masonry constructions, and LV3 method, which is the most refined analysis approach based on the use of FEM analysis programs. In the present paper the latter approach has been used to study the seismic behaviour of the Sidoni Palace, a monumental masonry building in the municipality of San Pio delle Camere damaged by the 2009 Abruzzo earthquake. In order to give validity to the achieved numerical results, experimental Ambient Vibration Tests (AVTs) have been performed on the examined palace with the purpose to identify its natural frequencies, corresponding modal shapes and damping coefficients. On the basis of the achieved numerical-experimental results, an effective retrofitting intervention has been designed and applied to the case study, its effectiveness being proved by further numerical analysis which will be used to address the future building rehabilitation.

2 The Abruzzo earthquake

On April 6th, 2009 at 3:32 a.m. (1.32 UTC) an earthquake generated by a normal fault, located in a valley contained between two parallel mountain along the direction North-South (Fanale et al., 2009), with maximum vertical dislocation of 25 cm and hypocentre depth of about 8.8 Km, stroke the Abruzzo region in the Central Italy (Indirli et al., 2013; Formisano et al., 2010).

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. This wag generated by many aftershocks (Figure 1) following the mainshock, which were recorded from accelerometer stations placed very close (4–5 Km) to the epicentre (Figure 2) belonging to the Italian Strong Motion Network (in Italian *Rete Accelerometrica Nazionale*, RAN), owned and maintained by the Department of Civil Protection (DPC).

Figure 1 Seismic sequence and location of the Abruzzo earthquake epicenter (see online version for colours)



Source: INGV

The Abruzzo earthquake is considered as an exceptional seismic action, since both the maximum recorded horizontal acceleration components within the epicentral area were larger than PGAs of the elastic spectra given by the current Italian Code (Ministerial

Decree of Infrastructure and Transport (M. D.), 2008) (Table 1) and the vertical acceleration effects were not negligible (near-field earthquake) (Monaco et al., 2009).

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, the region capital with about 73.000 people, where MCS intensity values changed between VIII and IX. Damages were even more significant in several villages located in the middle Aterno valley, where intensities as high as IX-X were experienced in Castelnuovo and Onna (Figure 3). In total, 14 municipalities experienced a MCS intensity between VIII and IX, whereas those characterised by MCS intensity 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 2 Elastic acceleration spectrum 4.3 Km far from the epicentre (FA030 station – site: Colle dei Grilli) (see online version for colours)

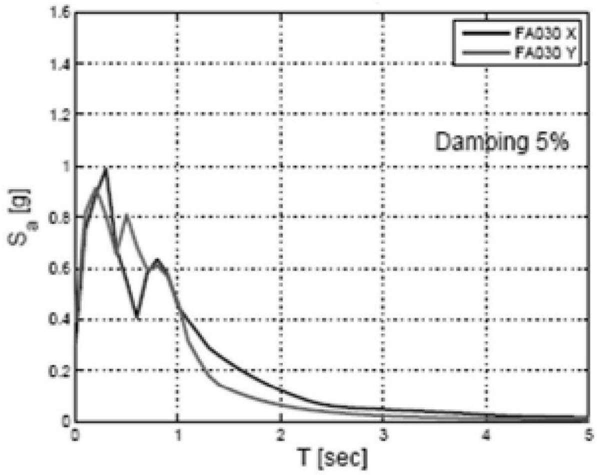


Table 1 Earthquake effects at different epicentre distances (ZPA (g) for grey coloured stations)

Record identifier	*Site*	*Dir. x*	*Dir. y*	*Dir. z*	*Epicentre distance* (Km)
		PGA (g)			
GX066	Aterno valley	0.626	0.597	0.420	4.8
FA030	Colle dei Grilli	0.416	0.434	0.215	4.3
CU104	Aterno river	0.394	0.451	0.380	5.8
AM043	Aquila parking	0.342	0.340	0.350	5.6
EF021	Assergi	0.153	0.149	0.112	18.0
TK003	Celano	0.081	0.089	0.045	31.6

3 The Sidoni Palace

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, where all the streets are orthogonal each to other. The whole rectangular area identified by these streets has dimensions of 70×56 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).

Figure 3 The historical centre of Castenuovo of San Pio after the Abruzzo earthquake (see online version for colours)

Before the earthquake, the historical centre mostly consisted of 2–3 stories ordinary buildings characterised 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 was observed in the buildings at the hill toe as respect to the constructions located on the hilltop. This aspect is due to some factors related to the topographic amplification, which have contributed to the strong shaking at the highest elevation of the village.

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 (Figures 4 and 5) is a monumental building in the old medieval nucleus of Castelnuovo. It is an isolated masonry palace, made of a sack stone masonry structure, having a regular and symmetric plan shape (Figure 6) and developing on two floors. The façade is also symmetric and characterised by regular openings and several architectural ornaments. The entrance is framed by an arch located in the façade central part.

The building ground floor develops on a rectangular surface of about 366 m^2 and is covered by tunnel vaults. The first level floor is composed of 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 7.

Figure 4 Bird's-eye view of the Sidoni Palace (circled in red) (see online version for colours)

Figure 5 View (a) and details (b) of the Sidoni Palace main façade (see online version for colours)

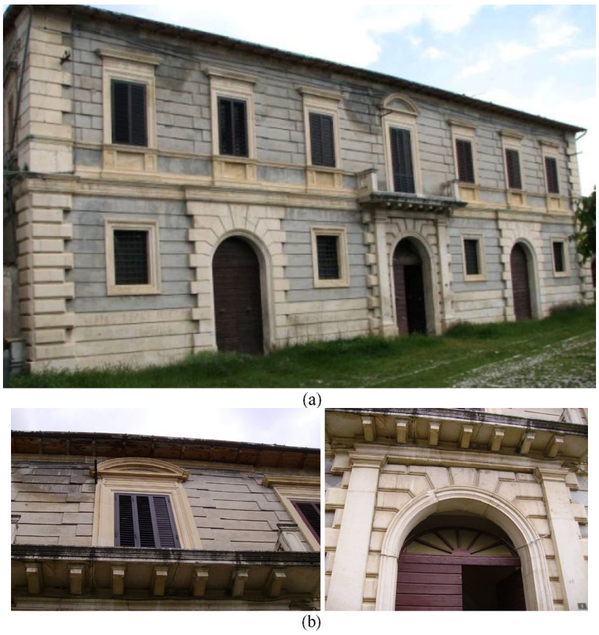
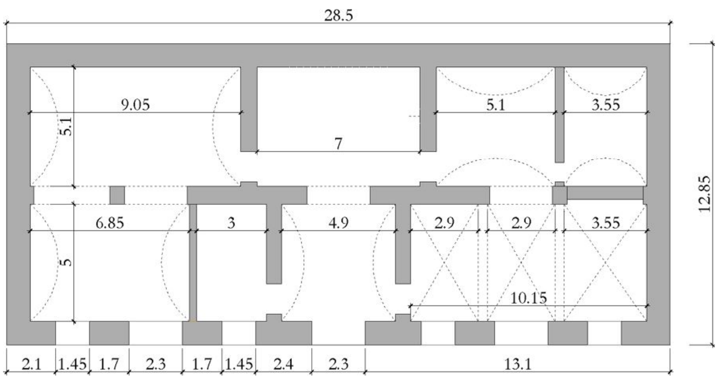


Figure 6 Ground floor layout of the Sidoni Palace



After earthquake no damage was recorded in the building facade, whereas significant damages and collapses of some masonry vault parts took place inside (Figure 8). Moreover, some of the steel floors at the building first level were destroyed by the earthquake, as illustrated in the building 3D views plotted in Figure 9.

Figure 7 Transversal section of the Sidoni Palace

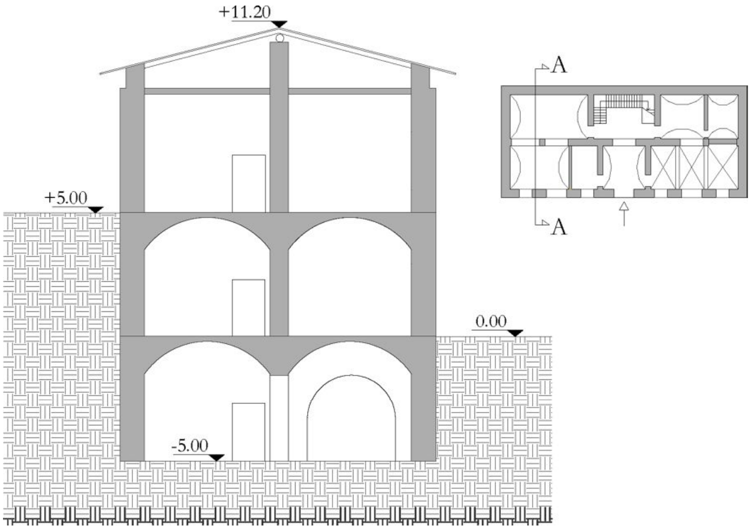
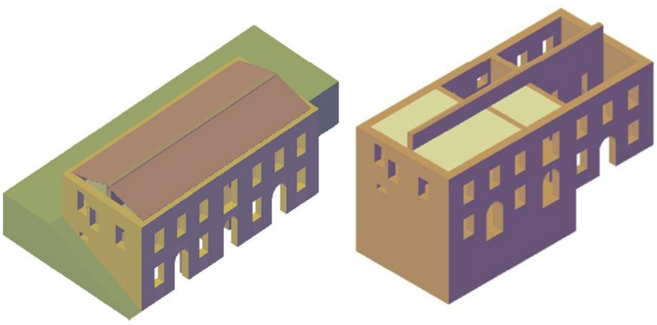


Figure 8 Damages occurred in the vaulted ceilings (see online version for colours)

Figure 9 3D views of the building (see online version for colours)

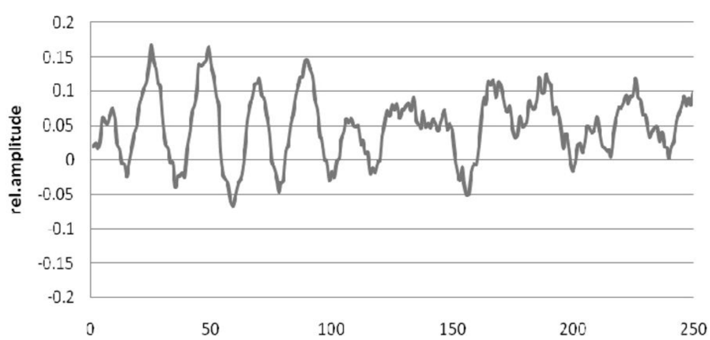


4 The experimental ambient vibration tests (AVTs)

4.1 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 (Figure 10). In particular, 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 as input signals.

Figure 10 Time history of vibrations excited by the ambient



The AVT is a non-destructive special investigation method purposely conceived especially for dynamic identification of buildings with historical and artistic importance (Pierdicca et al., 2016; Ubertini et al., 2016, 2017; Cavalagli et al., 2017; Clementi et al., 2017). It is a very fast analysis technique whose relatively simple implementation procedure can be applied to buildings in use, without disturbing their normal functioning.

The method basic assumption 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 vibration recording and record processing. The initial analysis phase is the dynamic calibration test. During this test all sensors, namely the seismometers, are placed in the same positions and directions. So, the signals are recorded simultaneously and the related Fourier spectra are obtained. The structure resonant frequencies can be preliminary defined using the dynamic calibration tests, but the natural frequencies final definition are achieved after the vibration mode shapes are gotten. After this calibration test, the seismometers are placed at different levels and points of the structure, but along the same direction, for simultaneous recording aiming at obtaining the vibration mode shapes. 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 the measurements into each monitored point are normalised as respect to the RP ones and, as a consequence, 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.

4.2 Testing equipment

During the ambient vibration measurements of Sidoni Palace, three seismometers Ranger type Kinematrics product were used (Figure 11) and the measured signal was amplified by four channel signal conditioner (Figure 12). The amplified and filtered signals from the seismometers were then collected by high-speed data acquisition system (Figure 13(a)), which transformed the analogue signals into digital ones. A personal computer (Figure 13(b)) and a special software for online data processing were used to plot the time histories of recorded velocities by the seismometers together with the response Fourier amplitude spectra (FAS) at any recorded point.

Figure 11 Equipment for ambient vibration measurements: the Ranger type seismometers (see online version for colours)

Figure 12 Equipment for ambient vibration measurements: the four channel signal conditioner (see online version for colours)

4.3 Post-processing and experimental results

Measurements of ambient vibrations on Sidoni Palace in Castelnuovo (Krstevska et al., 2010; Formisano et al., 2012) were performed along its transversal and longitudinal directions in selected points depicted in Figure 14 on the structure geometrical model generated by the ARTeMIS software based on the Tromino technology (Castellaro et al., 2016). The total number of the performed tests was 38, they including also the dynamic calibration tests. The datasets consisted on records of velocity signals with duration of 100 s and with a sampling frequency of 200 S_a/s.

Figure 13 Equipment for ambient vibration measurements: (a) high-speed data acquisition system and (b) personal computer for data processing (see online version for colours)

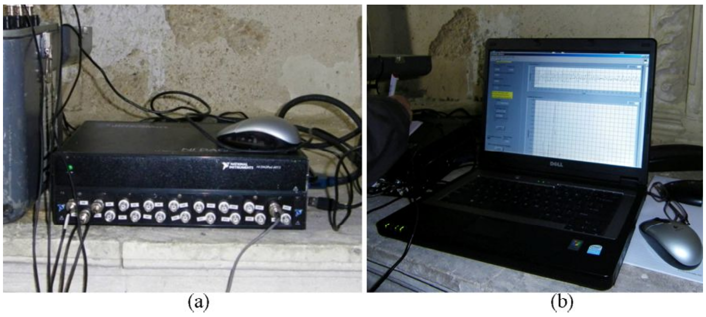
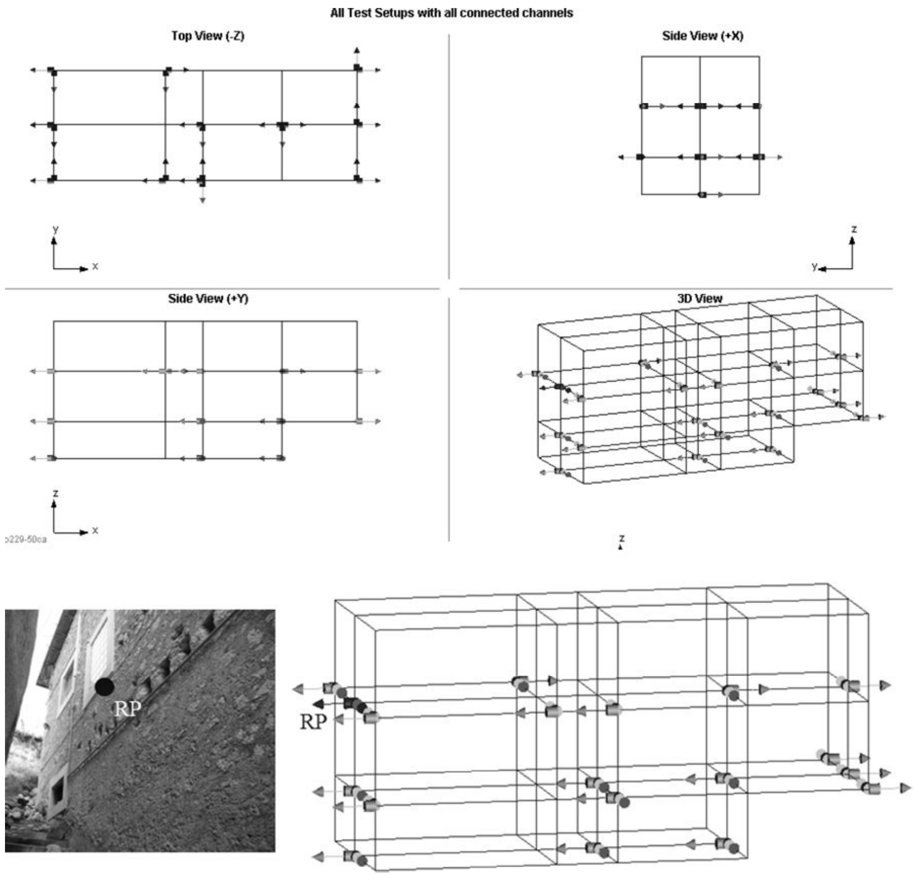


Figure 14 Top, side and 3D views of the palace geometry with positioning of the measuring points (RP is the blue point)



For post-processing and analysis of the recorded vibrations in all measuring points, the ARTeMIS extractor software was used (Structural Vibration Solutions A/S, 2010). 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 15. From this figure it is apparent that the spectrum is characterised by several dominant frequencies. The vibration shapes at particular frequencies are given in Figures 16–18. 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 clearly noticed.

Figure 15 Peak picking of the dominant frequencies of the Sidoni Palace

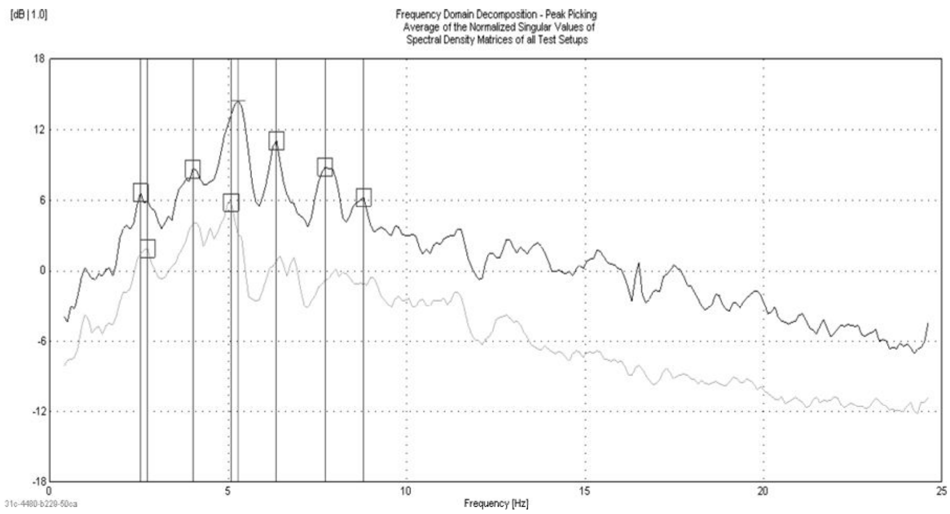
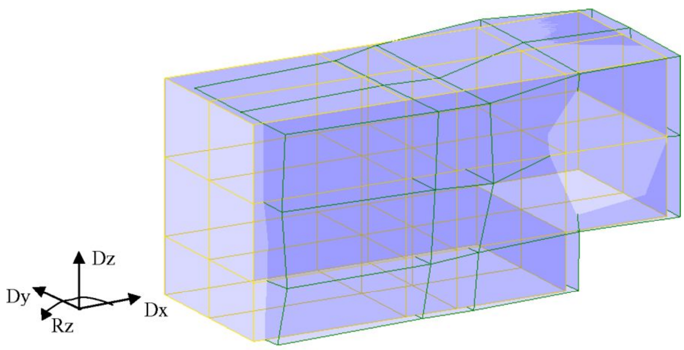


Figure 16 Vibration shape at a frequency of 5.08 Hz (transverse mode) (see online version for colours)



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%.

Figure 17 Vibration shape at a frequency of 5.27 Hz (longitudinal mode) (see online version for colours)

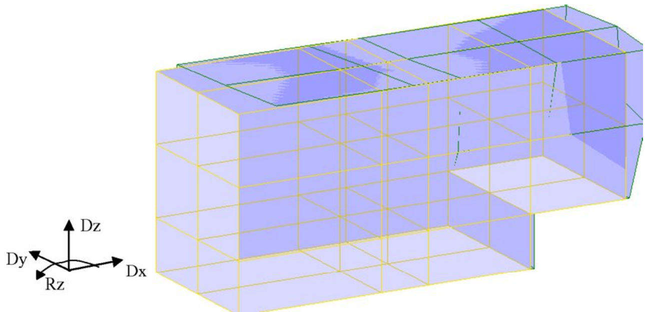


Figure 18 Vibration shape at a frequency of 6.35 Hz (torsion mode) (see online version for colours)

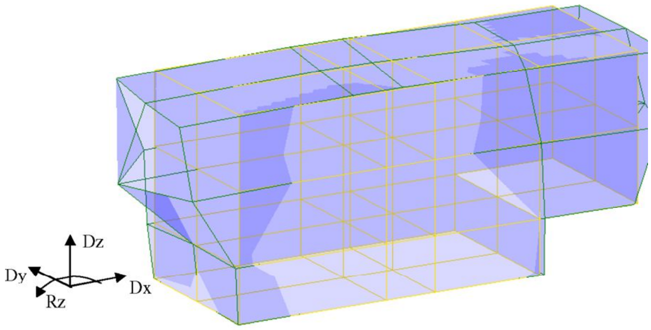


Table 2 Experimental frequencies and damping coefficients achieved from AVTs

Mode 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

5 The numerical activity

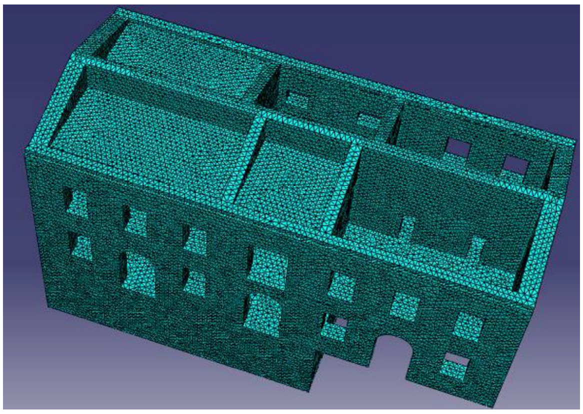
5.1 Calibration of experimental results

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

In order to properly assess the interaction among the building structural parts, the geometrical model has accurately reproduced all its main vertical and horizontal components, including openings, vaults and horizontal floors. With regard to this last aspect, it is worth to be precised that the collapsed floors have not been included in the model, in order to take into account the real building damage state (post-earthquake configuration).

The whole masonry structure, fully restrained at its base, has been discretised by means of tetrahedral 3D solid elements, that is C3D4 (4-node linear tetrahedron) elements (Figure 19).

Figure 19 The ABAQUS FEM model of the palace (see online version for colours)



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 (Ministerial Circular (M. C., 02/02/2009), 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 (Figure 20).

Figure 20 Flat jack test on masonry: test set-up (see online version for colours)

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

The experimental in-situ tests have provided the stress – vertical (Figure 21) and horizontal (Figure 22) strain laws.

Figure 21 Stress vs. vertical strain masonry's law

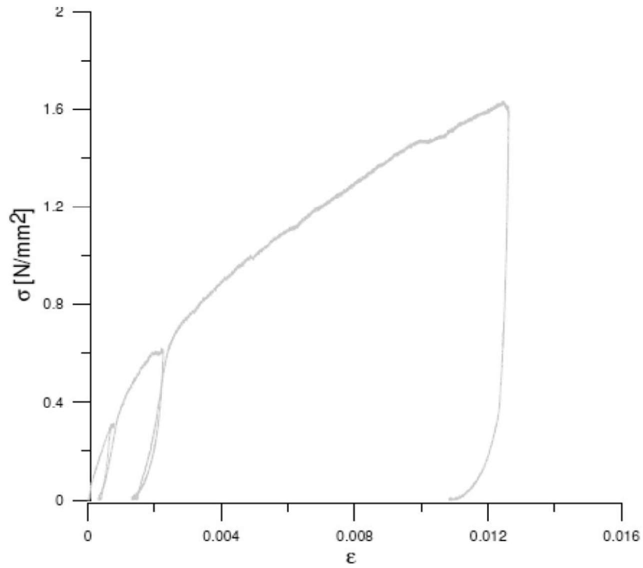
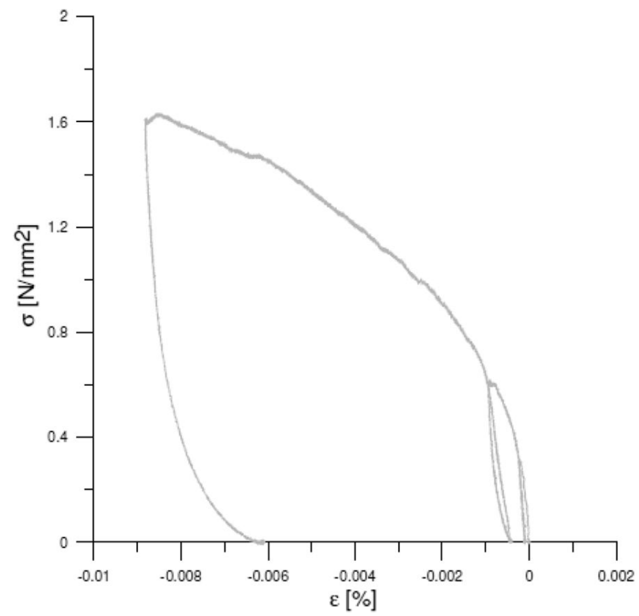


Figure 22 Stress vs. horizontal strain masonry's law



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 better approximating 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. It is worth of noticing that such a value is not within the elastic modulus range provided by the Italian M.C. for the considered masonry type.

Table 3 Numerical frequencies corresponding to different Young modulus values

	Mode frequency (Hz)	
E (MPa)	*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 could contain at least two stones or little more than two stones along its length, the following four mesh sizes have been considered:

- *Coarse mesh*: side length of 0.50 m
- *Medium mesh*: side length of 0.40 m
- *Fine mesh*: side length of 0.30 m
- *Very fine mesh:* side length of 0.20 m.

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 were 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 post-earthquake FEM model

	Transverse	*Longitudinal*
Mesh type	*f [Hz]*	*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 23–25. From these figures, it may be pointed out that, according to the real damages occurred due to the earthquake, the most deformable parts of the palace are masonry walls adjacent to the collapsed floors, whose constraining action is missed.

Figure 23 Transversal vibration mode (f = 5.06 Hz) of the actual building FEM model (see online version for colours)

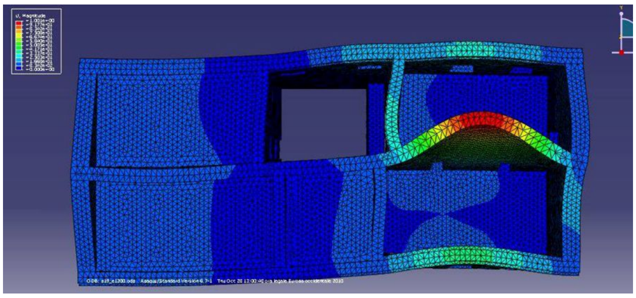


Figure 24 Longitudinal vibration mode (f = 5.17 Hz) of the actual building FEM model (see online version for colours)

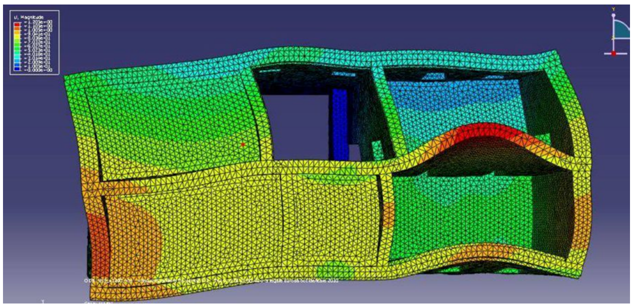
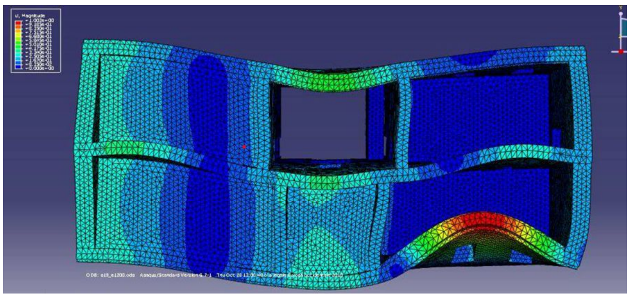


Figure 25 Torsion vibration mode (f = 6.59 Hz) of the actual building FEM model (see online version for colours)



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 found 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 Figures 16–18).

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

5.2 The original structure fem model

The frequency analysis has also been performed on the FEM model representative of the Sidoni Palace structural configuration before the April 6th seismic event (original configuration), in order to better investigate the structure seismic response.

The essential difference between the original configuration and the post-earthquake one is the presence in the first case of steel floors at the construction second level.

The modal vibration shapes of the original FEM model are shown in Figures 26–28. It may be noticed that the modal behaviour of this undamaged model (before the earthquake) is well defined in each directions, with no coupling of modes, as respect to the damaged model. Moreover, a noticeable difference of stiffness between the two implemented models is evidenced in Table 6.

Figure 26 Transversal vibration mode (f = 4.00 Hz) of the original structure (see online version for colours)

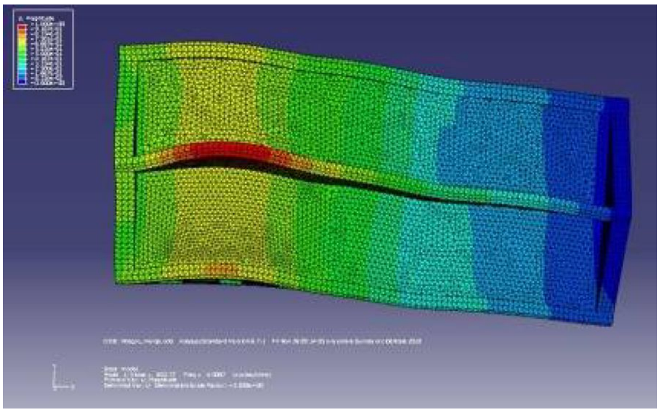


Table 6 Comparison among damaged FEM model frequencies and undamaged FEM model ones

Damaged model f [Hz]	*Undamaged model* f [Hz]	*Frequency decrease*[%]	*Stiffness decrease* [%]
3.92	4.00	2	4.0
5.17	5.32	3	6.1
6.59	7.09	7	14.5

Figure 27 Longitudinal vibration mode ($f = 5.23$ Hz) of the original structure (see online version for colours)

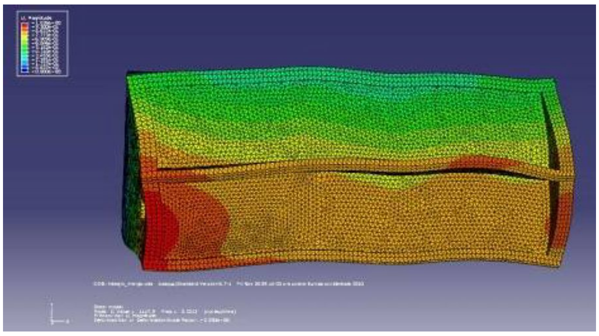
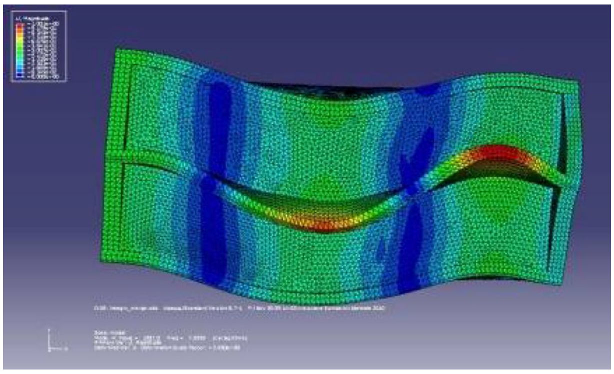


Figure 28 Torsion vibration mode ($f = 7.09$ Hz) of the original structure (see online version for colours)



In particular, the first transversal 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). This comparison reveals that the stiffness of the damaged model averagely decreases of the order of 4% with respect to the building one before earthquake.

5.3 Damage evolution curves

Finally, after the in-situ experimental identification of the structure dynamic properties, whose results have been successively confirmed by numerical analyses, the progressive damage of the Sidoni Palace during earthquake has been assessed by means of appropriate damage evolution curves. In fact, as the numerical models have shown, the reduction of the building natural frequencies is related to the stiffness decrease, which is a factor representative of the seismic damage. Therefore, a simplified damage indicator ($d_{k,i}$) based on the variation of the natural frequencies due to the damage level may be expressed by the following formulation (Mendes et al., 2010):

$$d_{ki} = 1 - \left(\frac{f_{k,i}}{f_{k,0}}\right)^2 \tag{1}$$

where

f_{k0}: natural frequency of the mode shape k before the earthquake;

$f_{k,i}$: natural frequency of the mode shape k after the earthquake.

The damage evolution curves along both plane directions of the building have been drawn through the graphic relationship between the number of modes (abscissa) and the simplified damage indicator d_k (ordinate), whose values are listed in Table 7 (Figure 29). From these curves it is apparent that:

- the damage is greater in longitudinal direction (*X*);
- the major damage occurs between 1st and 2nd vibration modes in direction *X* and between 2nd and 3rd ones in direction *Y*;
- after these damages, the increase of the damage indicator (d_{ki}) is almost constant in both directions.

Figure 29 Damage evolution curves of the Sidoni Palace under earthquake (see online version for colours)

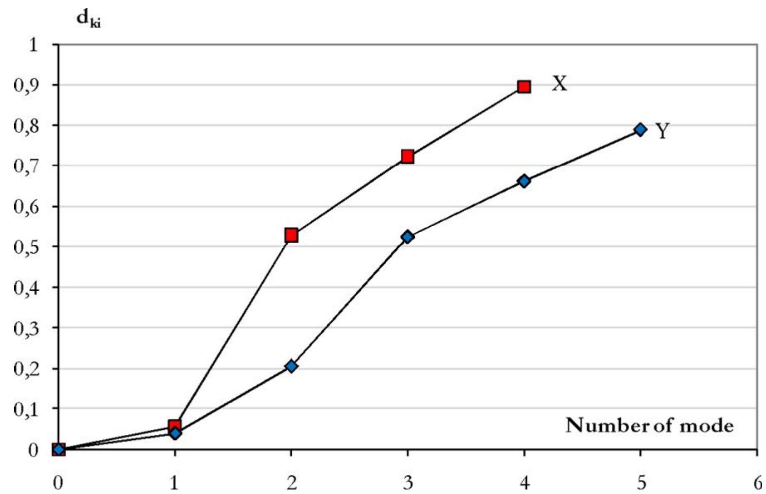


Table 7 Simplified damage indicator d_{ki} of damage evolution curves

Dir.	f_{ki} [Hz]	f_{k0} [Hz]	d_{ki}
(*X*)	5.17	5.32	0.055596
	6.59	9.08	0.528853
	8.26	9.2	0.722761
	8.79	9.66	0.894774
(*Y*)	3.92	4	0
	5.06	5.54	0.0396
	5.85	7.09	0.205378
	7.21	7.76	0.524579
	8.07	8.64	0.661308

5.4 The retrofitted fem model

The analysis results have shown the high structural vulnerability of the Sidoni Palace, mostly due to the low effectiveness of wall-to-wall and floor-to-wall connections. Thus, on the basis of the performed analyses, retrofitting measures have been proposed for the restoration of the examined construction.

At first, the rebuilding of the collapsed floors and the strengthening of existing ones with the creation of r.c. slabs have been proposed as floor consolidation interventions. In fact, r.c. slabs allow to create rigid diaphragms, which assure an adequate distribution of seismic forces among all of the bearing walls. Moreover, these floors have been well connected to the walls by means of the arrangement of r.c. tie beams.

Second, the repair of the damaged vaults and the placement of metal ties have been considered. 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 box-type behaviour of the entire structure. Moreover, this technique allows to avoid possible out-of-plane overturning mechanisms of masonry walls (Tomazevic et al., 1993; Applied Technology Council (ATC-43), 1998).

The metal tie intervention technique herein proposed consists of metal bars inserted at the vault bases along 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 wooden roof consolidation through the arrangements of perimeter tie beams.

Taking into account all the aforesaid interventions, a retrofitted FEM model of the Sidoni Palace has been still implemented by means of the ABAQUS code. In particular, the constraint conditions among the consolidated 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 FEM model aiming at modelling the metal ties at the vault bases to connect masonry walls each to other.

Afterwards, frequency analyses have been performed on this retrofitted model, providing the modal shapes depicted in Figures 30–32.

Figure 30 Transversal vibration mode (f = 4.15 Hz) of the retrofitted FEM model (see online version for colours)

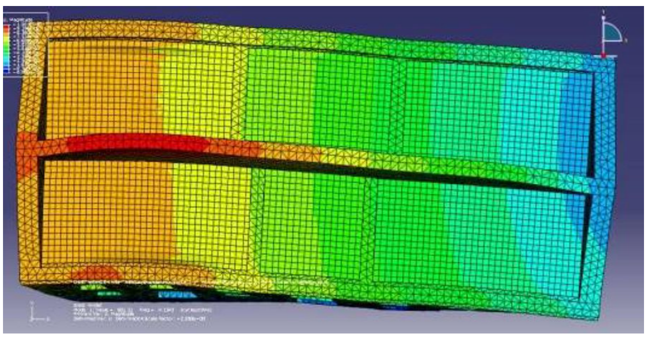


Figure 31 Longitudinal vibration mode ($f = 5.37$ Hz) of the retrofitted FEM model (see online version for colours)

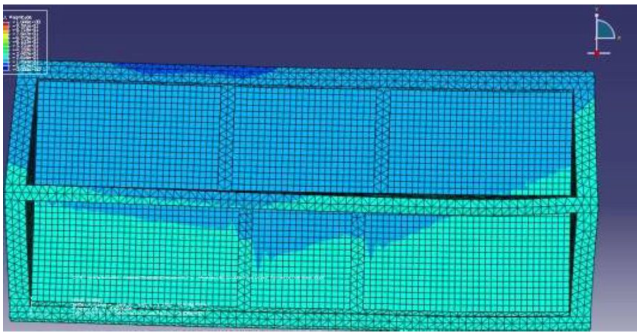
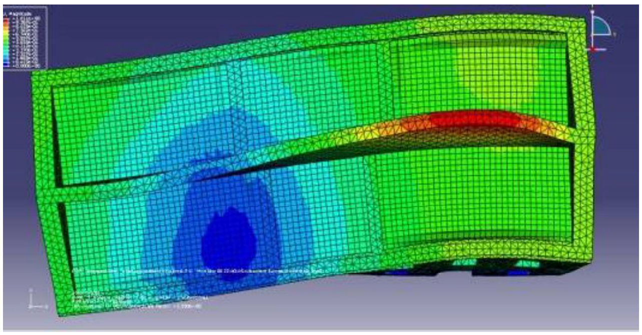


Figure 32 Torsion vibration mode ($f = 6.41$ Hz) of the retrofitted FEM model (see online version for colours)



From the previous figures and according to the results depicted in Table 8, it is noticed that the modal behaviour of the structure is drastically changed, since global modal shapes are initial ones and the local modes coincide with superior ones. In addition, the structural stiffness increases of the order of 12% both for the transverse and longitudinal fundamental vibration modes. Definitively, the modal behaviour of the building is totally improved.

Table 8 Comparison between the damaged FEM model and retrofitted one in terms of dominant frequencies

Damaged f [Hz]	*Retrofitted* f [Hz]	*Frequency increase* [%]	*Stiffness increase* [%]
3.92	4.15	6	12.36
5.17	5.37	6	12.36
6.59	9.21	28	63.8

5.5 Vulnerability curves development

All the achieved numerical results have allowed to plot the vulnerability curves of the Sidoni Palace for each of the three examined structural configurations (undamaged, damaged and retrofitted), by means of the following relationship (Mendes et al., 2010):

$$d^*_{ki} = \frac{1}{\left(\frac{f_{k0}}{f_I}\right)^2 - 1} + d^*_{ki-1} \tag{2}$$

where:

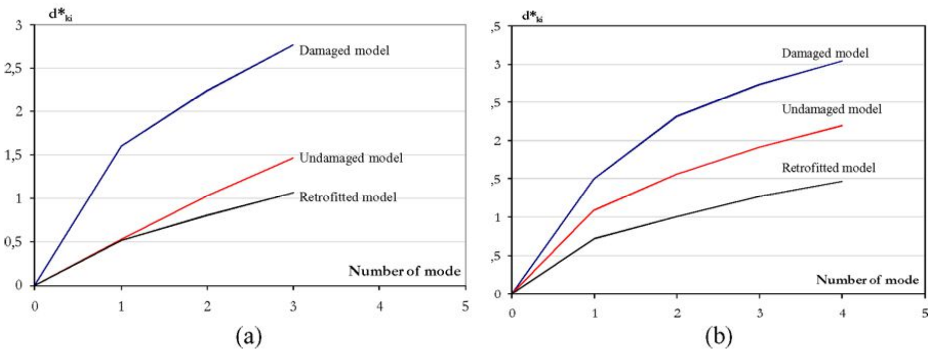
f_k: natural frequency of the k-mode

f_I: natural frequency of the first mode

d^*_{ki}: damage indicator of the (k–1) mode.

In Figure 33 the damage indexes of the building along both the longitudinal (X) and the transverse (Y) directions have been plotted as a function of the mode number.

Figure 33 Vulnerability curves of Sidoni Palace in directions X: (a) and Y (b) (see online version for colours)



In the longitudinal (X) direction, the achieved curves have shown that:

- the d^*_{ki} factor is larger for the building damaged from the earthquake
- the damage suffered by undamaged and retrofitted buildings is similar in the first mode
- the efficiency of retrofitting interventions is detected starting from the 2nd vibration mode
- in all cases, damage is reduced as the number of mode increases.

On the other hand, in the transverse (Y) direction it is apparent that:

- the damaged building and the retrofitted one suffers major and minor damages, respectively
- the undamaged building shows damages of intermediate level as respect to the others
- in all cases, the major damage occurs in the first mode and the damage is reduced as the number of mode increases
- the reduction of the d^* factor with mode number is less evident as the structural behaviour improves.

6 Conclusions

In the paper, experimental and numerical activities performed on a monumental construction hit by the Abruzzo earthquake are presented.

The results of experimental ambient vibration tests have shown that in the domain from 0 Hz to 25 Hz several frequencies are predominant. In particular, the clearest frequencies in the transversal direction and in the longitudinal one are 5.09 Hz and 5.27 Hz, respectively. The tests have also provided the equivalent damping coefficient values, they ranging from 2.1% to 5.6%.

The experimental results have been used for the analytical investigation of the Sidoni palace. Therefore, FEM models of the building have been implemented by means of the ABAQUS non-linear numerical code, considering the structure configurations before and after the earthquake. This investigation has permitted to evaluate the building stiffness reduction after earthquake, equal to 4% and 14.5% for the first mode and the third one, respectively, as well as to define appropriate damage evaluation curves of the palace. These curves allowed to declare that:

- the damage is greater in longitudinal (*X*) direction
- the major damage occurs between 1st and 2nd vibration modes in direction *X* and between 2nd and 3rd ones in direction *Y*, whereas after these damages the damage increase is almost constant in both directions.

Finally, appropriate retrofitting interventions, namely strengthening of existing floors with the creation of r.c. slabs connected to the walls by means of r.c. tie beams, repairing of damaged vaults with placement of metal ties at their bases, replacement of fractured brackets, e relocation of fallen stones and joints filling and wooden roof consolidation through arrangement of perimeter tie beams, have been applied to the damaged palace. The effectiveness of these interventions has been proved by performing numerical analyses on a palace improved FEM model. Thanks to these interventions, the modal behaviour of the retrofitted structure has been drastically improved with a substantial increase of the structure natural frequencies, corresponding to a significant stiffness augment, equal to 12.36% for the longitudinal and transverse modes and to 63.8% for the torsion mode. This last step has been useful to develop the building vulnerability curves for each examined direction and structural condition, they being able to evaluate the structure damage evolution as the number of vibration modes increase. From the achieved curves it has been noticed that:

- in both analysis directions damage is reduced as number of mode increases
- in direction *Y* the major damage occurs in the first mode
- retrofitting interventions in direction Y are more effective than the direction X ones.

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