

## DYNAMIC TESTS ON AN EXISTING R.C. SCHOOL BUILDING RETROFITTED WITH “DISSIPATIVE TOWERS”

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**Abstract.** *This paper deals with the dynamic tests carried out on a school building (four story reinforced concrete frame) seismically retrofitted with an innovative system that uses external steel dissipative towers. Before the retrofitting, ambient vibration tests were carried out with the aim of evaluating the actual linear dynamic of the building including the contribution of non-structural components (e.g. external and internal walls). Modal parameters determined by means of experimental tests are crucial for the final design of the retrofitting system allowing the calibration of a predicting f.e. model. Three low noise servo-accelerometers per floor were opportunely positioned at each floor to monitor its rigid motion. The modal parameters are identified with the Enhanced Frequency Domain Decomposition technique obtaining the natural frequencies, mode shapes and damping ratios. After the building retrofitting, snap-back tests at different load levels were performed in order to evaluate the dynamic characteristics of the new structural system. The load was applied in a quasi-static manner by means of two Dywidag bars  $\phi 47$  anchored at the last floor and connected to a steel triangular truss pulled by two hydraulic jacks; the instantaneous release was obtained by cutting a dog-bone shaped steel plate with a blowtorch. In addition to the accelerometers, displacement transducers were positioned at the base of the steel towers, close to the viscous dampers, to measure the motion of the two towers. The natural frequencies and the damping ratios of the retrofitted building are estimated from the time histories of accelerations (free decay functions) by means of the crossing time and the logarithmic decrement techniques, respectively. The adopted experimental methodology adopted has revealed to be effective for the dynamic characterization, both at very low strain and at higher strain level, of a low-rise reinforced concrete frame building in service, which is characterized by a high overall stiffness due to the contribution of the non-structural components.*

## 1 INTRODUCTION

In the last decades considerable efforts have been made by researchers in the development and improvement of methodologies to identify the dynamic properties of civil engineering structures by means of experimental tests.

Various testing techniques can be used which differ for various aspects, such as equipments, time-consuming, costs, and dynamic input. Depending especially on the input amplitude, some methods allow investigating the dynamic response of buildings only in the elastic range, other both in the elastic and inelastic range. Among those of the first class, one of the most attractive methods of measuring the dynamic characteristics of real buildings is the ambient vibration testing which uses natural vibrations (e.g. micro tremors, wind, anthropic activities noise) without requiring any artificial input action. The advantages of this method are that small, light, and very portable instrumentation are required and that tests can be carried out without disrupting the service of the building. On the other hand, the method requires the use of specific low noise accelerometers capable of measuring very low amplitude vibrations. Because of the low amplitude range of the ambient vibrations ( $10^{-5}$  g), dynamic characteristics evaluated with this method may be different from those obtained from strong-motion ( $> 0.1$  g) records due to non-linear effects [1-5]. To evaluate the dynamic characteristics under higher amplitude, different tests can be performed such as the stepped and sweep sine test, the release test (snap-back or free vibration test) or vibration tests induced by blast loading.

This paper deals with the dynamic tests on an existing school building seismically retrofitted with an innovative patented system [6] based on the structural coupling of the building with new external steel towers equipped with dissipative devices. After a description of the building and the retrofitting system, the equipments and the testing procedures are presented. Tests were performed before and after the retrofitting works. Ambient vibration tests were carried out before the retrofitting with the aim of identifying the dynamic properties of the building and of calibrating the predictive model for the retrofit design. In particular, dissipative towers are designed so that the building remains in the elastic range without any damage, undergoing very small displacements (lower than the limit imposed by the Italian Standard [7] for the condition of immediate occupancy for the case of the Life Safe earthquake intensity). This means that the retrofitting system must be designed considering the actual overall initial stiffness of the building, i.e. the stiffness due to the contribution of structural and non-structural components. After the retrofit, snap-back tests on the building-towers system were carried out with the aim to identify and evaluate changes of the dynamic properties due to the retrofitting system [8]. Finally, a comparison between the experimental and numerical modal parameters of the structure before the retrofitting is presented with the aim to demonstrate that the vibration test and the data processing techniques used allow to identify the dynamic characteristics of r.c. buildings in service.

## 2 DESCRIPTION OF THE BUILDING AND RETROFITTING SYSTEM

The High School B. Croce in Avezzano town, not far from L'Aquila (Italy), is a 4-story r.c. building constructed in the 60's, which needed to be seismically retrofitted to meet the recent Italian seismic regulations [7]. An innovative system for the retrofit of existing buildings has been adopted, based on the coupling of the structure with new steel truss towers erected externally and equipped with dissipative devices (Dissipative Towers). This methodology enables to carry out retrofitting works without interrupting the activities inside the building.

The building is composed of 3 main 4-story blocks (A, G, and D) placed around a 1-story block (C-AM); other two 1-story blocks (B and D) are located laterally to block D. Figure 1 shows a plan view of the entire building with the dissipative towers whereas Figure 2a and b

illustrate an elevation section and a lateral view of the block A, respectively. This block, interested by the experimental tests, is a 4-story r.c. frame with a plan dimension of about  $13.6 \times 47.8$  m; the first floor is located about 1.3 m above the ground level, the inter-story height is 3.5 m and the last floor has a medium height of about 1.5 m. The concrete frame structure has 2 span of 6.6 m and 2.8 m in the transverse direction and 12 spans of 3.9 m in the longitudinal direction. Columns have  $300 \times 600$  mm cross sections, oriented in the transverse direction; while beams in the transverse and longitudinal directions are different; in particular, transverse beams have  $300 \times 600$  mm cross sections whereas longitudinal beams have  $300 \times 450$  mm or  $450 \times 160$  mm cross sections.

The seismic retrofitting of the building has been obtained with six external dissipative towers connected to each floor, excluding the first one (Figures 1 and 2). The main blocks A and G are protected with two steel towers per block, located at the back side. The blocks B, D and F, originally separated by expansion joints, are protected with two dissipative towers which reciprocally connect the three blocks (Figure 1). Towers protecting blocks A and G have been used to locate a lift (TA) and an emergency stairwell (TS). Each tower is erected on a r.c. thick base plate that is centrally pinned to the foundation plate by means of a spherical support. Eight dissipative devices (viscous dampers) are located between the base plate and the foundation plate (2 devices per vertex) so that the rigid rotation of the base plate, due to the horizontal displacement of the building, activates simultaneously all the devices. The dampers are inserted into an articulated quadrangle (Figure 2c) that significantly amplifies the device displacements due to the rotation of the tower base.

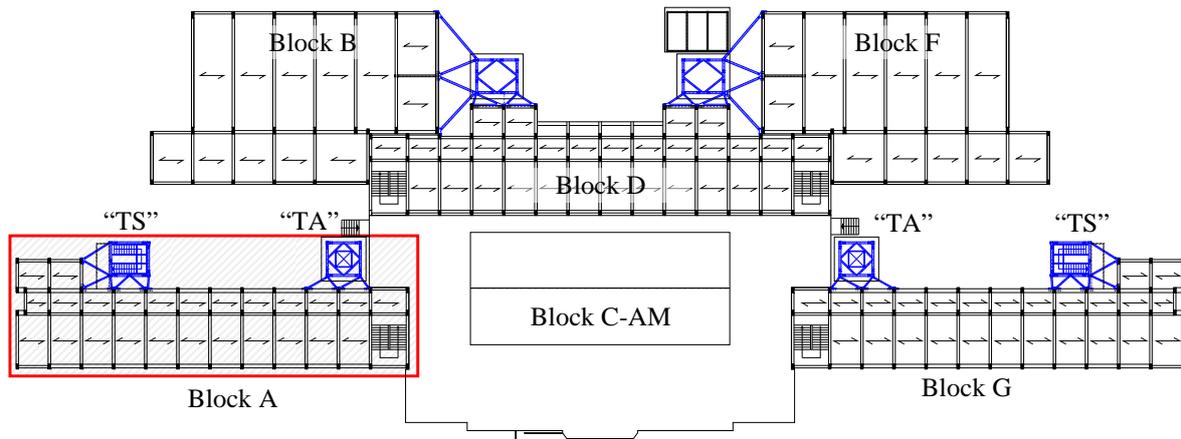


Figure 1. First floor plan view.

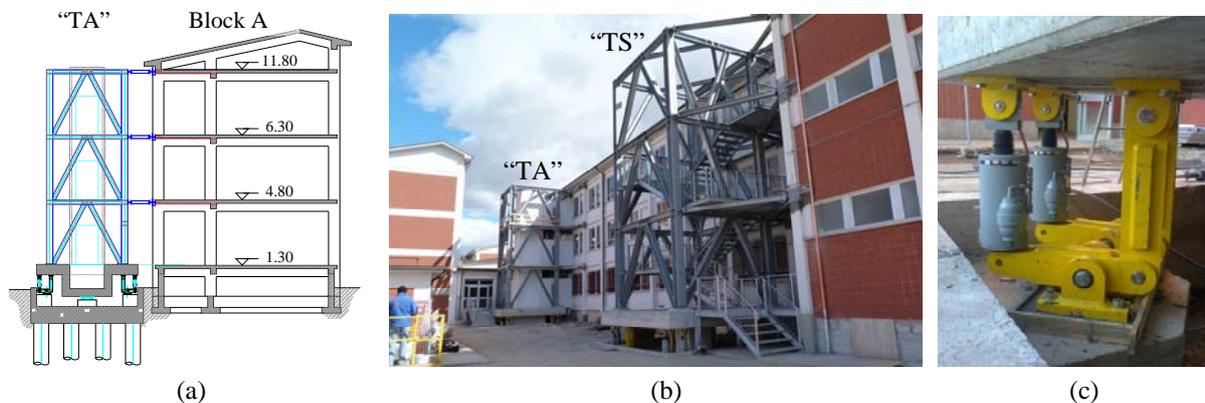


Figure 2. (a) Elevation section; (b) lateral view of block A with dissipative towers; and (c) dissipative devices.

### 3 TESTS AND DATA PROCESSING METHODS

Two different dynamic tests were carried out, the first before the seismic retrofitting and the second after the works. Ambient vibration tests were performed on the blocks A and G to obtain the modal parameters of the actual building and snap-back tests only on the block A to identify changes in the dynamic properties of the building as a result of the retrofitting works.

#### 3.1 Tests before retrofitting

The ambient vibration tests were carried out using a 24-bit data acquisition system connected to 14 low noise servo-accelerometers by means of coaxial cables. Four different tests were performed by varying the sampling frequency (from 250 to 1000 Hz) and the time of acquisition (from 1 to 20 minutes). Three accelerometers per floor were positioned: two sensors, measuring along two orthogonal axes (transverse and longitudinal), were placed in the same point at a side of the building while the third, measuring along the transverse direction, was located in the opposite side of the building, far from the first two, to better catch the rotational component of the floor. Other two sensors were placed at the ground floor.

The modal parameters are extracted by means of the Enhanced Frequency Domain Decomposition (EFDD). Firstly, the recorded data are suitably processed by applying a baseline correction, by filtering in the frequency range 0.1-25 Hz, and finally by downsampling at 50Hz. The EFDD, which is an extension of the Frequency Domain Decomposition, allows estimating damping ratios, in addition to the natural frequencies and mode shapes [9]. According to this technique, working in the frequency domain, modes are extracted by simply peak picking from the Singular Value Decomposition of the spectral densities of accelerations. For each peak, a frequency band is selected evaluating the MAC value between the shape relevant to the peak and to the neighbour frequencies, while all the rest is set to zero. The Power Spectral Density function is taken back to the time domain using the Inverse Discrete Fourier Transform obtaining an approximation of the correlation function of the SDOF system. The natural frequencies and the damping ratios are obtained by means of the crossing time and the logarithmic decrement methods, respectively [10].

In Table 1 the frequencies and the relevant damping ratios obtained from experimental data are listed. The first three natural frequencies are quite close to each other and can be associated to the first two translational modes (in longitudinal and transverse directions) and to the rotational mode; the fourth natural frequency is higher and corresponds to an in-plane distortional mode. The relevant damping ratios seem to be quite low; this may be due to the closeness of frequencies that may lead to underestimate the damping ratios, evaluated with the EFDD procedure. Different techniques for the estimation of damping ratios may be used (e.g. global like SSI) and the authors are still working in this direction.

#### 3.2 Tests after retrofitting

After the retrofitting two different tests were performed on block A: Static (S) and Static followed by Snap-Back (S+SB). Both tests were carried out varying the maximum load applied. The test type and the maximum load applied for each test are listed in Table 2.

The load was applied in a quasi-static manner by means of 2 Dywidag  $\phi 47$ , placed horizontally and anchored to the last floor; these bars were connected to the top vertex of a triangular steel truss pinned to ground at the one vertex at the base and pulled by 2 hydraulic jacks at the remaining vertex (Figure 3a). For the snap-back test, once the release load was reached, the truss was blocked with a dog-bone shaped steel plate (Figure 3b) and the quick release was obtained by cutting the steel plate with a blowtorch.

Mode	Frequency [Hz]	Damping ratios [%]	Mode type
1 <sup>st</sup>	5.25	0.53	1 <sup>st</sup> longitudinal
2 <sup>nd</sup>	5.56	1.78	1 <sup>st</sup> transversal
3 <sup>rd</sup>	6.16	2.42	1 <sup>st</sup> rotational
4 <sup>th</sup>	10.43	0.65	in-plane distortional

Table 1. Frequencies and damping ratios estimated from the vibration test.

Test	Date	Type	Maximum Load [t]
Test_1	29.04.2012	S + SB	188
Test_2	30.04.2012	S + SB	94
Test_3	30.04.2012	S	117
Test_4	30.04.2012	S + SB	141
Test_5	04.05.2012	S	141
Test_6	04.05.2012	S + SB	160

Table 2. Tests performed after retrofitting.

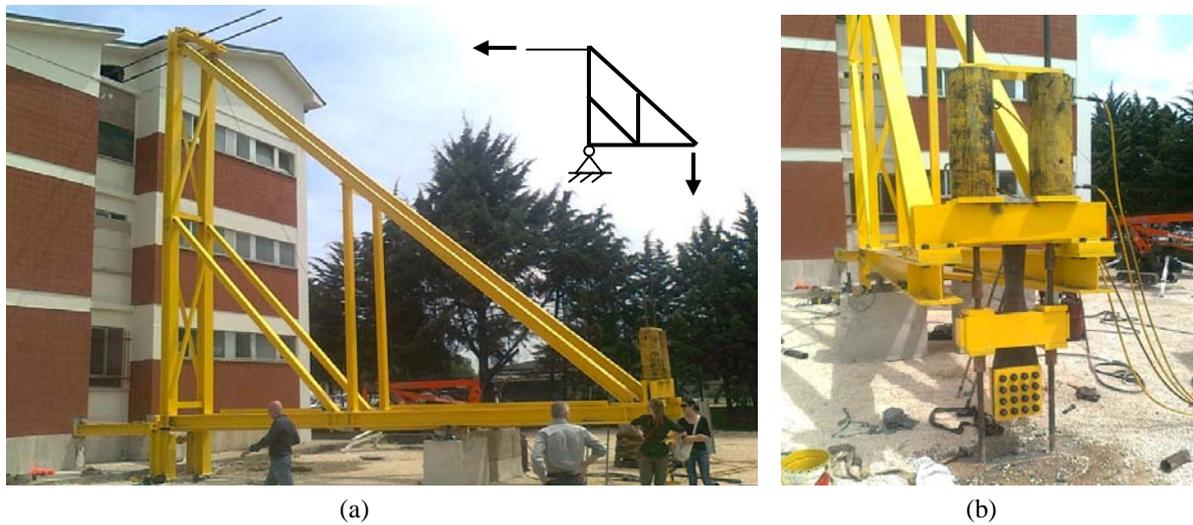


Figure 3. Loading system: (a) triangular steel truss; (b) hydraulic jacks and dovetail shaped steel plate.

The measuring chain consisted of a 24-bit data acquisition system with 12 channels, 9 uni-axial piezoelectric accelerometers, 4 displacement transducers (2 dynamically and 2 statically sampled) and coaxial cables. Three accelerometers per floor (excluding the ground floor) were positioned by adopting the same configuration of the ambient vibration tests. Two displacement transducers per tower were placed near the dampers at two corners of the base plate of the tower, and positioned vertically. Considering that the displacement of the central point of the base plate is null and assuming that the plate behaves as a rigid body, the rotation of the base of the tower can be simply deduced from the vertical displacement component of two points of the plate.

Figure 4 shows the time histories recorded by the accelerometers after two snap-back tests. In Test\_1 (Figure 5a), characterized by the maximum applied horizontal load, a maximum acceleration of about  $1/10$  g has been measured in Y direction (accelerometer AY), parallel to the imposed displacement, while lower values have been registered in the transverse direction (accelerometers AX and BX). It is interesting to note that a decrement in time of the amplitude is clear for AY, with a trend similar to that of a damped harmonic oscillation while in transverse direction, especially for BX, the acceleration seems to increase after the release.

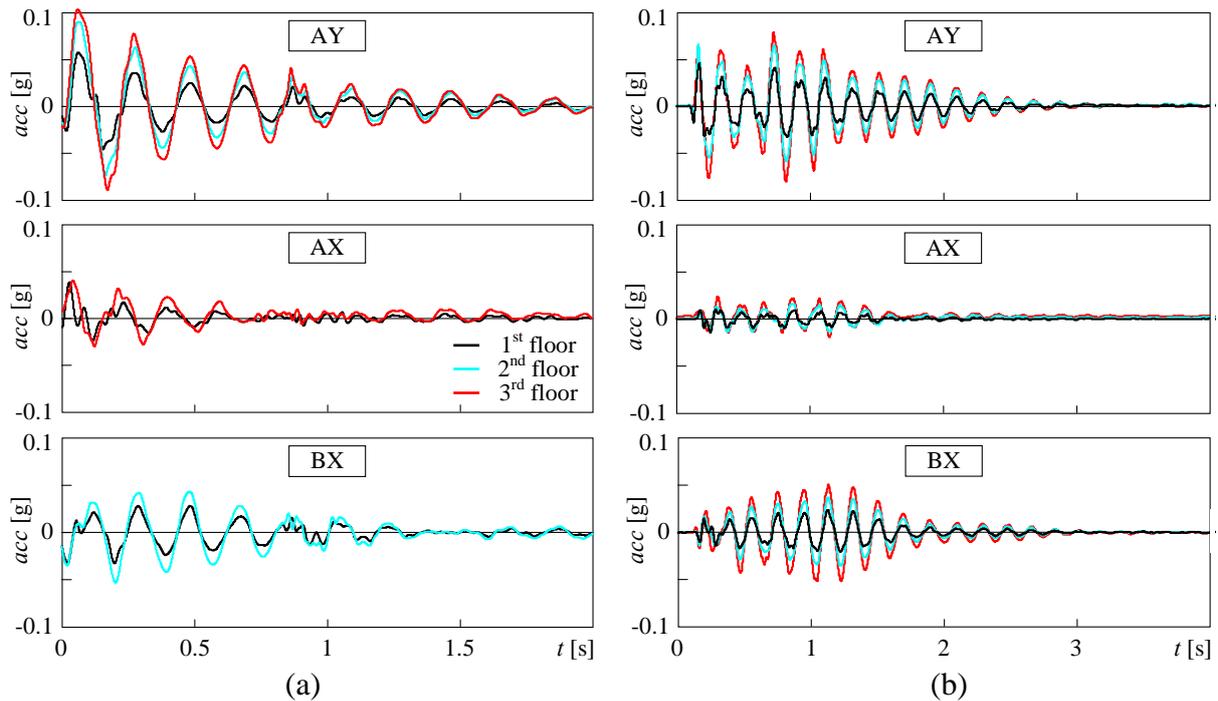


Figure 4. Time histories of accelerations: (a) Test\_1; (b) Test\_2.

This behaviour is due to the fact that the building is pulled along Y with a force slightly eccentric with respect to the centre of rigidity of the building leading to an initial oscillation of roto-translational type. During the oscillations the rotational component increases with respect to the longitudinal one leading to an increase of the transverse displacement. In the Test\_2 (Figure 5b) the building response is less clear than in the Test\_1; a similar behaviour is obtained also from Test\_4 and Test\_6 which are not reported in this paper for the lack of space. This may be due to the fact that the maximum load applied is sensibly lower in these cases than in Test\_1. For such low levels of load, the displacement imposed to the structure is too small to permit a perfect coupling between the building and the dissipative towers because of some gaps and the non perfect contact at the connection systems (bolted connections) among building, towers, and dampers.

In Figure 5 the displacement recorded during Test\_1 by the displacement transducers positioned below the base plate of the two towers are shown. Dotted lines refer to the transducers at the base of the tower TA which have been statically sampled, while continuous lines refer to those of the tower TS, dynamically sampled. On the left side of the figure the force vs. displacement graph regarding the static part of the test is reported. The absolute values of the displacements relevant to the tower TS are sensibly different whereas the values relevant to the tower TA are almost the same. This is mainly due to the horizontal displacement component of the building, greater for tower TA than for tower TS, due to the counterclockwise rotation of the building which is induced by the eccentricity of the applied load with respect to the centre of rigidity of the building. The right side of the figure regards the snap-back test; the free damped oscillation of the building after the quick release is clear and also the presence of a unique frequency contribution to the oscillation seems to be evident. The small residual displacement (offset from the zero) observed for one transducer is likely due to the non perfect connections among building, towers and dampers.

From the free decay of accelerations the natural frequencies and damping ratios can be estimated. The frequencies are obtained by making a linear regression of the crossing times of the decay function, while the equivalent viscous damping ratios are estimated with a linear

regression evaluating the logarithmic decrement from the extremes of the function, i.e. peaks and valleys. The procedure, implemented in a Matlab routine, allows repeating the computation considering an increasing number of peaks, evaluating changes in the dynamic properties of the coupled system during the oscillation. This procedure may give some information on the stability of the results and on the variability of the dynamic characteristics during the oscillation due, for example, to a non linear behaviour of the system that may occur for large oscillation amplitudes.

The natural frequencies are obtained from signal AY of all the dynamic tests carried out, whereas the damping ratios are estimated only from Test\_1 since a free decay is not clear in the other cases. For all tests, in Table 3 the frequencies obtained by considering a different number of the extremes are listed; the “final” value is obtained considering only few extremes extracted from the final part of the signal. By increasing the number of extremes involved in the computation, natural frequencies tend to the “final” values; by considering just few extremes more scattered values are obtained. Furthermore, by considering final values, a mean value of about 5.18 Hz is obtained; this is slightly smaller than 5.25 Hz, obtained from ambient vibration test. The period elongation after the building retrofitting may be due to the fact that the coupling of the building with the towers leads to an increase of the total mass of the system; with regard to the stiffness, the towers provide an increase of the stiffness for the superior modes but not for the firsts.

The damping ratios obtained from signal AY of Test\_1, by considering a different number of extremes, are listed in Table 4. The damping decreases by increasing the number of considered peaks and the final value, obtained by considering one peak of the final part of the signal (with a very low oscillation amplitude), is even a little bit smaller but greater than that obtained before the retrofitting with the ambient vibration test.

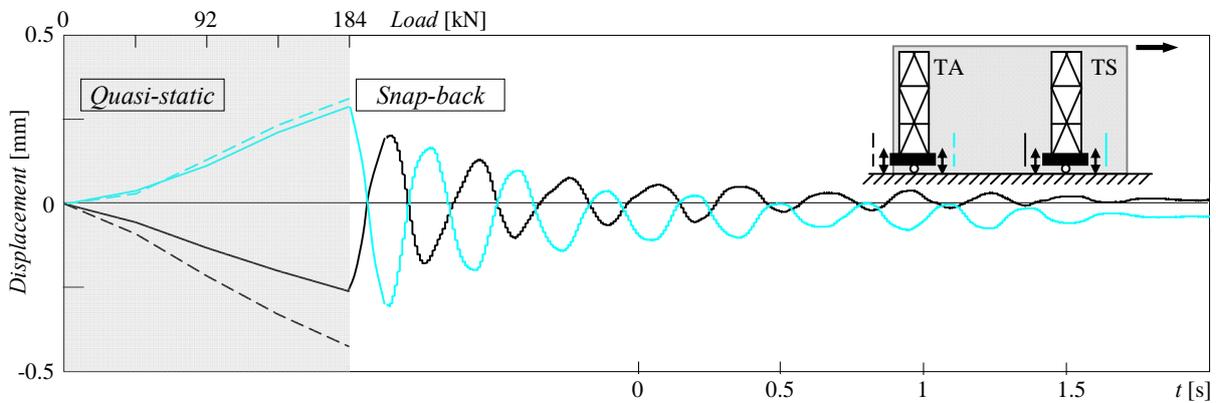


Figure 5. Time histories of displacements at the base of towers during Test\_1.

N. of peaks	2	4	6	8	10	12	14	16	18	20	22	Final
Test_1	4.98	4.87	4.88	4.97	4.93	4.97	5.01	5.05	5.06	5.08	5.08	<b>5.20</b>
Test_2	7.73	6.35	5.76	5.53	5.45	5.44	5.32	5.28	5.28	5.326	5.25	<b>5.18</b>
Test_4	7.64	6.27	5.67	5.97	5.94	5.70	5.51	5.43	5.38	5.36	5.35	<b>5.21</b>
Test_6	6.22	5.15	5.15	5.05	4.95	4.92	4.99	5.07	5.12	5.14	5.13	<b>5.12</b>

Table3. Frequencies estimated from the snap-back tests after the retrofitting.

N. of peaks	2	4	6	8	10	12	14	16	18	20	22	Final
Test_1	5.47	5.93	5.20	4.79	5.05	4.67	4.31	4.20	4.24	4.34	4.34	<b>3.60</b>

Table 4. Damping ratios estimated from the snap-back tests after the retrofitting.

This behaviour may be attributed to the contribution of the dissipative towers which is greater for greater amplitude of oscillation and tends to disappear for very low amplitudes because of nonlinearities of the system due, for example, to gaps at the connections. Unfortunately, because of some technical problems on the loading system, tests at higher level of force could not be carried out. However, further investigations are needed to draw some general conclusions.

#### 4 NUMERICAL MODEL

A finite element model is developed by means of SAP2000 code [11] representing the building before the retrofitting. The structural elements such as beams, columns and plates, are modelled with elastic beam and shell elements. In order to match adequately the actual behaviour of the building registered from the experimental tests, the non-structural components have been modelled since their contribution cannot be neglected at very small strains. Light walls and external walls are thus modelled with shell elements and the stiffness contribution of non-structural components of the floors is suitably taken into account in the definition of the floor shell elements. Furthermore, the tangent elastic modulus is considered for the material.

The geometry is obtained from design drawings and technical surveys, whereas material properties are estimated from an experimental characterization, for concrete, and from the technical literature, for masonry. The tangent elastic modulus of the concrete is evaluated with

$$E = V_p^2 \rho \frac{(1+\nu)(1-2\nu)}{(1-\nu)} \quad (1)$$

where  $V_p$  is the P-wave velocity,  $\rho$  is the mass density and  $\nu$  the Poisson ratio.  $V_p$  and  $\rho$  were estimated from core samples extracted by the structures. In particular, the ultrasonic pulse velocity method was used for evaluating the P-wave velocity. Finally, a Poisson ratio equal to .2 was assumed, as recommended by NTC 08 for uncracked concrete. The same value of elastic modulus is used for the floor. The elastic modulus of the light internal masonry is estimated starting from the results of the studies carried out by Nichols and Totoev [12] that find, for pressed red clay bricks, an elastic modulus and a Poisson's ratio of about 14000 N/mm<sup>2</sup> and 0.22, respectively. Assuming the internal wall constituted of hollow bricks with a percentage void volume of about 65 %, as usual in Italian buildings, and neglecting the mortar, a value of 4900 N/mm<sup>2</sup> was obtained and used as initial tentative value for the light internal masonry. Considering the variability of masonry, the elastic modulus was modified step by step to optimize the model, trying to capture the fourth natural frequencies experimentally identified; the procedure leads to a value of 5600 N/mm<sup>2</sup>. In view of the particular configuration of the windows rigid braces are on the bottom part of the perimeter columns are used to simulate the external walls below the windows. Properties used for materials are summarized in Table 5.

The vibration frequencies and the relevant mode shapes are evaluated by means of an eigenvalue analysis; Figure 6 shows the first four mode shapes and the corresponding frequencies. The comparison between experimental and numerical frequencies shows a good agreement with a maximum difference of 2.16 % (Table 6).

	Elastic modulus [MPa]	Poisson ratio	Mass density [t/m <sup>3</sup> ]
Concrete	31601	0.2	23
Masonry	5700	0.22	6.8

Table 5. Material properties.

Test	Experimental	FEM	Difference [%]	Mode type
	Frequency [Hz]	Frequency [Hz]		
1 <sup>st</sup> mode	5.25	5.24	-0.19	1 <sup>st</sup> longitudinal
2 <sup>nd</sup> mode	5.56	5.44	2.16	1 <sup>st</sup> transversal
3 <sup>rd</sup> mode	6.16	6.29	-2.11	1 <sup>st</sup> rotational
4 <sup>th</sup> mode	10.43	10.52	0.86	in-plane distortional

Table 6. Comparison between the natural frequencies obtained experimentally and with FEM.

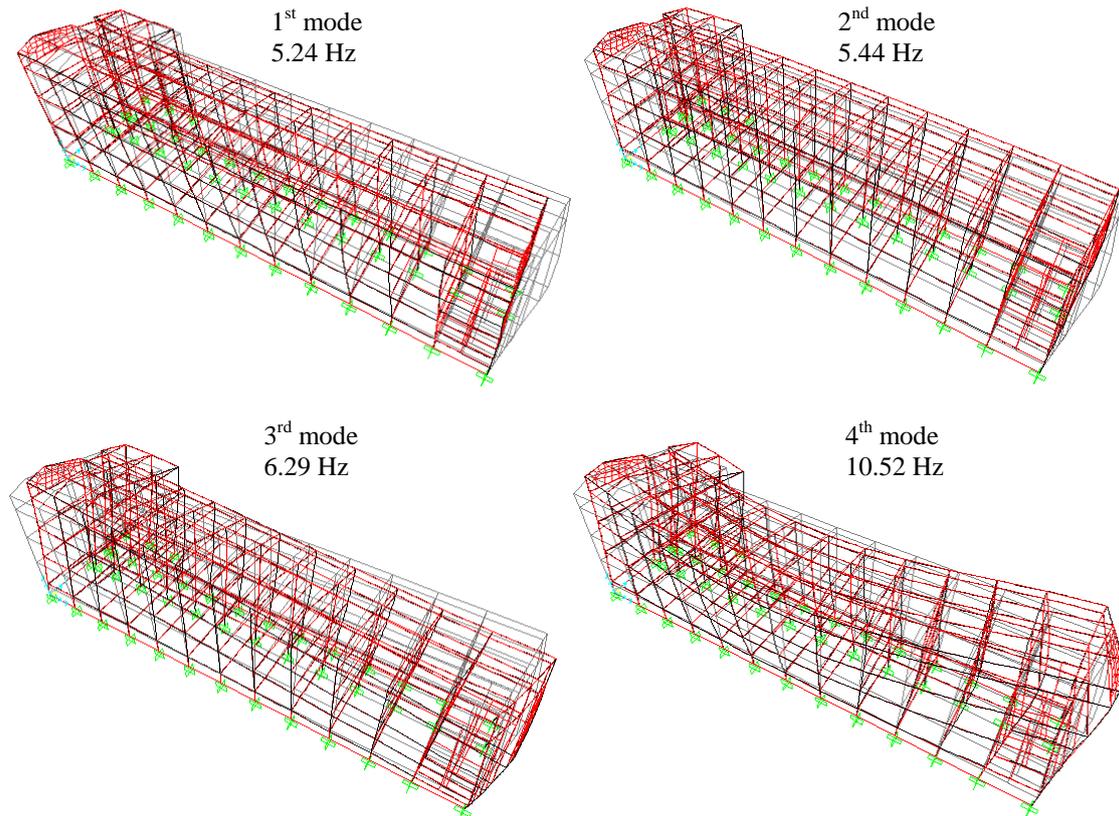


Figure 6. Mode shapes obtained with FEM

## 5 CONCLUSION

The results of a series of dynamic tests, performed before and after the seismic retrofitting with dissipative towers, on a school building has been presented. Ambient vibration test were carried out on the structure before the retrofitting while snap-back tests with different load levels were performed on the retrofitted structure. An experimental modal analysis of the accelerometer signals allowed estimating the natural frequencies, damping ratios and mode shape. From the comparison between the modal parameters, before and after the retrofitting works, a slight elongation of the period and an increase of the damping ratios were observed.

Finally, the experimentation was effective to calibrate a finite element model of the building taking account of the overall stiffness due to both structural elements and non structural components. Numerical simulations of the snap-back tests are currently under study.

## REFERENCES

- [1] S.S. Ivanovic, M.D. Trifunac, M.I. Todorovska, Ambient vibration tests of structures - A review. *ISET J. Earthq. Tech.*, 37(4), 165-197, 2000.
- [2] F. Dunand, J.E. Rodgers, A.V. Acosta, M. Salsman, P.-Y. Bard, M. Çelebi , Ambient vibration and earthquake strong-motion data sets for selected USGS extensively instrumented buildings, *Open-File Report 2004-1375*, 2004
- [3] M. Turek, C. Ventura, S. Guerrero, Ambient vibration testing and model updating of a 44-storey building in Vancouver, Canada, *25th International Modal Analysis Conference*, Orlando FL, USA, Paper 250, 2007.
- [4] C. Michel, P.Guéguen, S. El Arem, J. Mazars, P. Kotronis, Full-scale dynamic response of an RC building under weak seismic motions using earthquake recordings, ambient vibrations and modelling, *Earthquake Engineering Structural Dynamics*, **39**, 419-441, 2010.
- [5] C.S. Oliveira, M. Navarro, Fundamental periods of vibration of RC buildings in Portugal from in-situ experimental and numerical techniques, *Bulletin of Earthquake Engineering*, **8**, 609-642, 2010.
- [6] A. Balducci. Dissipative Towers. Application n. EP20100747238 20100831, WO2010EP62748 20100831, International and European classification E04H9/02 – Italian concession n° 0001395591, 2005
- [7] D.M.14.01.2008, Nuove Norme Tecniche per le Costruzioni, Ministero delle Infrastrutture, *G.U. n.29, 04.02.2008* (in Italian).
- [8] J. Rodrigues, M. Ledesma, Modal identification of a viaduct before and after retrofitting works, *25th Int. Modal Analysis Conference*, Orlando, FL, USA, 2007.
- [9] R. Brincker, C.E. Ventura, P. Andersen, Damping estimation by frequency domain decomposition, *19th International Modal Analysis Conference*, Kissimmee, FL, USA, pp.698-703, 2001.
- [10] N.J. Jacobsen, P. Andersen , R. Brincker, Eliminating the influence of harmonic components in operational modal analysis, *25th Int. Modal Analysis Conference*, Orlando, FL, USA, 2007.
- [11] SAP2000 advanced (v14.1.0) Static and dynamic finite element analysis of structures, Berkeley, *CSI Computer & Structures, Inc.*, 2009.
- [12] J.M. Nichols and Y.Z. Totoev, Experimental determination of the dynamic modulus of elasticity of masonry units, *15th Australian Conference on the Mechanics of Structures and Materials*, Melbourne. Vic., Australia, 1997.