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Procedia Engineering 199 (2017) 2084–2089

Procedia Engineering

www.elsevier.com/locate/procedia

X International Conference on Structural Dynamics, EURODYN 2017

Dynamic behaviour of a retrofitted school building subjected to the after-shock sequence of the 2016 Central Italy earthquake

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Abstract

This paper deals with the dynamic behaviour of a 1960s school building in Camerino subjected to moderate earthquakes belonging to the after-shock sequence of the 2016 Central Italy earthquake. The school structure, constituted by RC frames, was seismically retrofitted by means of "Dissipative Towers", an innovative system based on the use of external stiff steel truss towers equipped with dissipative dampers. Modal properties of the building after the retrofit have been determined through ambient vibration tests. After the main shock of the 2016 Central Italy earthquake, the building was instrumented with low-noise accelerometers and the dynamic behaviour of the structure during several aftershocks was recorded. Firstly, a numerical finite element model of the building is developed and calibrated on the basis of the ambient vibration measurements; then the registered dynamic response of the structure for the highest aftershock event (among the registered ones) is compared with that obtained numerically with the calibrated model. Results demonstrate the good agreement of the registered and predicted response.

Peer-review under responsibility of the organizing committee of EURODYN 2017.

Keywords: Dissipative towers; dynamic monitoring; modal parameters; operational modal analysis; earthquake aftershocks

1. Introduction

Dynamic identification is a technique increasingly used in civil engineering and particularly in existing buildings for the calibration of a reliable structural model of the building (e.g. to be used for design of repairs, rehabilitation and retrofit works), and for the structural health monitoring. This paper deals with the dynamic monitoring of a 1960s RC frame school building in Camerino that was instrumented with low-noise accelerometers few days after

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Peer-review under responsibility of the organizing committee of EURODYN 2017.
10.1016/j.proeng.2017.09.478

the main shock of the 2016 Central Italy earthquake. In particular, the registered structural behaviour, in terms of accelerations and displacements, is compared with that obtained from a refined finite element model of the building, developed for the seismic retrofit, which is achieved through the use of "Dissipative Towers" [1]. The retrofit system is based on the structural coupling of the building with new external steel towers equipped with dissipative devices. A refined model of the building was calibrated on the basis of ambient vibration measurements carried out after the retrofit works with the aim of identifying the dynamic properties of the building.

After a brief description of the building and the retrofitting system, the refined finite element model, including non-structural components (i.e. floors, internal partitions, external walls), calibrated on the basis of ambient vibration measurements, is presented. Then, the permanent dynamic monitoring system is described and the registered dynamic response of the structure for the highest aftershock event (among the registered ones) is compared with that obtained numerically with the aim to show the capability of the model to predict the dynamic behaviour of the real building under moderate earthquakes.

2. Description of the school building

The school building is located in Camerino, a small town near the epicentre of the earthquake of magnitude 6.0 that struck the Central Italy in August 2016. The structure has overall dimensions of 26.0 × 48 m and is composed by three different blocks (B1, B2 and B3) constituted of RC frames, juxtaposed with simple expansion joints. Fig. 1a, b reports the plan view and sections of the building with the position of the expansion joints (red dashed lines): B1 and B2 are constituted by 4-storeys RC frames while B3 is formed by a 3-storeys RC frame, all with an inter-storey height of 3.8 m. Frames are characterized by columns of square cross section (40 × 40 cm), rotated by 45° with respect to the frame planes, and by linearly tapered beams, on the building perimeter, with a cross section of about 30 × 80 cm at the beam-column joint and 30 × 40 cm at mid-span. Internal beams have a uniform rectangular cross section with different dimensions, depending on the beam position. Joints separating B1 and B2 only concerns structural elements whereas non-structural components, such as screeds floorings and infill walls, are continuous. On the other hand, joint separating B2 and B3 concerns both structural and non-structural elements.

The seismic retrofit of the school has been achieved by means of two steel truss towers placed externally to the building. The towers, positioned as shown in Fig. 1a, are rigidly connected at all floor levels, excluding the first one (Fig. 1b, c), by means of steel braces that are fixed to steel plates anchored to the external beams of the frame structure. Furthermore, B2 and B3 have been structurally connected by means of thick steel plates anchored to the RC frame elements. It should be noted that the presence of joint separating B1 and B2 was not known when the retrofit was designed as it was hidden by the continuity of non-structural elements. Each tower is erected on a RC thick base plate that is centrally pinned to the foundation basement through a spherical support. Eight dissipative viscous dampers are located in vertical position between the base plate and the foundation basement and connected through a leverage system that allows amplifying the device displacements due to the rotation of the tower base.

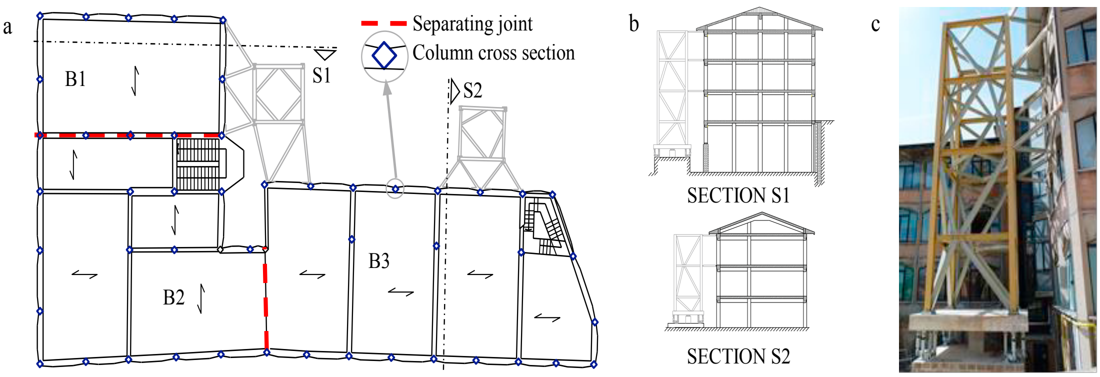

Fig. 1. (a) Plan view; (b) sections of the school building; c) one of the installed tower

The dissipative system is dimensioned so that the building remains in the elastic range without any damage, undergoing very small displacements, even for the case of the Life Safe earthquake intensity [2]. Consequently, the retrofitting system has to be designed considering the actual overall initial stiffness of the building, i.e. the stiffness due to the contribution of both structural and non-structural components. In this sense, ambient vibration tests were particularly useful to calibrate a model able to describe the actual behaviour of the building in its actual service conditions.

3. Modal parameters of the building before the main shock

After the retrofit, ambient vibration tests, based on the Operational Modal Analysis [3] technique, have been performed (May 2013) to evaluate the modal parameters of the retrofitted building. Ambient vibration measurements were carried out with low noise piezoelectric accelerometers connected to a 24-bit data acquisition system by means of coaxial cables. Three accelerometers per floor were positioned: two sensors were placed in point A at a corner of the building (Fig. 4), measuring along two orthogonal directions (Ax and Ay, in transverse and longitudinal directions of the building, respectively) while the third was located at the opposite side of the building in point B (Fig. 4), measuring along the transverse direction (Bx). In the case of floors behaving rigidly in their plane, the configuration allows capturing the rigid motion of each storey. The last floor, being not accessible, was not monitored so that 9 accelerometers were placed. For each configuration, 500 s long records sampled at a rate of 2048 Hz (conditioned by the equipment) were acquired; the record length provides enough data to obtain modal parameters with a good accuracy, since the acquired time window follows between 1000-2000 times the fundamental structural period of the building [3]. Table 1 contains the modal parameters of the first seven modes of the building obtained by the Enhanced Frequency Domain Decomposition (EFDD) and the Covariance-driven Stochastic Subspace Identification (SSI-COV) [4], i.e. the average values of frequencies f and damping ratios ξ, with the relative standard deviations Δf and $\Delta\xi$. As for damping ratios ξ, identified values, ranging between 0.09 and 1.40%, are more scattered than frequencies with a standard deviation between 0.17 and 1.18%. However, the obtained values are in the range of expected values for RC frame structures.

Finally, a 3D isometric view of the seven identified mode shapes is reported in Fig. 2. It is worth noting that last floors of B1 and B2 are not included since they were not monitored. The first three modes are those typical for low-rise buildings and can be assimilated to a first transversal mode with torsional component, a first torsional mode, and a first longitudinal mode with torsional component, respectively. The torsional component is distinctly present in all cases due to the L-shaped plan of the building (assuming that also B1 and B2 behave monolithically for low intensity actions).

Table 1. Modal parameters before the main shock

	EFDD				SSI-COV			
Mode	f (Hz)	Δf (Hz)	ξ (%)	$\Delta\xi$ (%)	f (Hz)	Δf (Hz)	ξ (%)	$\Delta\xi$ (%)
1	3.60	0.01	1.40	0.17	3.59	0.02	1.39	0.10
2	3.84	0.01	0.27	0.40	3.81	0.01	1.55	0.38
3	4.16	0.02	0.98	0.25	4.13	0.02	1.61	0.37
4	5.00	0.03	0.77	0.27	4.97	0.02	1.31	0.12
5	7.69	0.17	0.09	0.84	7.66	0.03	1.28	0.21
6	8.70	0.11	0.23	0.79	8.68	0.04	5.09	1.50
7	9.54	0.27	1.09	1.18	9.63	0.10	2.17	0.75

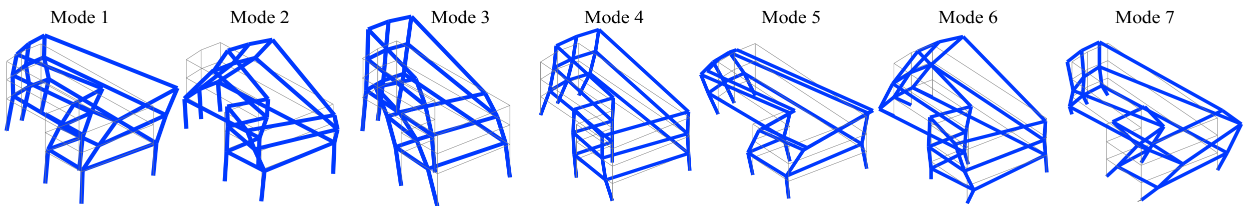

Fig. 2. Identified mode shapes

4. Finite element model

A finite element model of the building is developed by means of SAP2000 code [5] to interpret results of the ambient vibration tests. Structural components such as beams and columns are modelled with elastic frame elements while floors and walls (both light internal partitions and external walls) are simulated with shells (Fig. 3a).

Different Young's moduli of elasticity have been used for the material constituting structural and non-structural elements. Young's modulus of structural concrete elements (E_{cs} = 30180 MPa) such as beams, columns and floors, are based on an experimental survey, while Young's modulus of walls is chosen according to the Italian Standards [2], considering hollow brick masonry with a percentage of void volume of less than 45% for the external masonry (E_{cs} = 2000 MPa) and in the 45-65% range for the internal ones (E_{cs} = 1200 MPa). At the foundation level, springs are adopted to simulate the soil-structure interaction. The numerical model is calibrated according to the sensitivity method [6, 7] by varying mechanical parameters of materials in the neighborhood of above mentioned values. In Table 2 the first four natural frequencies obtained from the numerical model are compared with the experimental ones while in Fig. 3b the comparison between numerical and experimental mode shapes is graphically reported through the Modal Assurance Criterion (MAC). According to this criterion, a MAC equal to 1 identifies the perfect matching of the experimental and numerical mode shapes while a MAC equal to 0 denotes the orthogonality of the two modes. MAC values are also reported in Tab. 2. It is worth noting that the developed model is able to well reproduce the experimental data, both in terms of frequencies and mode shapes.

5. School dynamic behaviour subjected to an aftershock event of the Central Italy Earthquake (August 2016)

The earthquake of August 24th, 2016, was well resisted by the structure that only underwent minor damage to a limited number of internal claddings. Soon after the earthquake, the building was instrumented with low noise piezoelectric accelerometers connected to a 24-bit data acquisition system by means of coaxial cables. A continuous monitoring of the building was scheduled from 27/08/2016 to 29/08/2016. In the sequel labels #Ax, #Ay and #Bx will be adopted to indicate registrations of sensors placed at different floors of the building, from the foundation level (floor -1) to the top (floor 2).

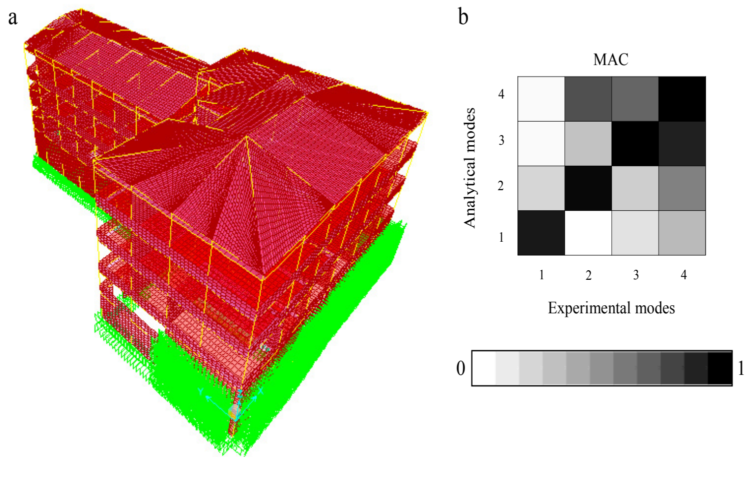

Fig. 3. a) Finite Element Model of the building; b) MAC between analytical and experimental mode shapes

Table 2. Natural frequencies obtained by Analytical model and Experimental tests

Mode	Experimental (EFDD) (Hz)	Analytical (Hz)	MAC
1	3.60	3.42	0.97
2	3.84	3.88	0.96
3	4.16	4.23	0.94
4	5.00	5.40	0.86

The earthquake of the 28/08/2016 of magnitude 4.2, with epicenter in Ascoli Piceno was registered by the monitoring system.

Fig. 5a shows the time histories of accelerations registered by sensors -1Ax, -1Ay and -1Bx at the foundation level of the building while Fig. 5b shows the time histories of the relevant displacements, obtained by double integration of the acceleration time series. It is worth noting that accelerations registered at point B are higher than those resulting at point A. This can be explained by recognizing that foundations of B3 rest on the ruins of old walls on the perimeter of the block (Fig. 5a); thus, the structure is overall less stiff on the side of B3. In addition, Fig. 6a, b shows time histories of displacements obtained from the integration of the relevant accelerations registered by sensors located at the first and second level of the schools, respectively.

The available numerical model of the building is used to numerically evaluate the registered response; to this purpose, accelerations registered at the foundation level are used as input motion for the numerical applications; since in the transverse direction two registrations are available (-1Ax and -1Bx), the one relevant to point A is considered, by assuming a synchronous input motion.

Fig. 6a, b shows with yellow lines the numerical response of the building in terms of displacements obtained at the location of the sensors at floors 1 and 2, through the developed finite element model. Comparisons of predicted and measured displacements once again demonstrate the reliability of the developed model in predicting the building dynamic response. According to the moderate intensity of the registered earthquakes and taking into account the very moderate damage underwent by the building to internal claddings, a refined model including external walls and internal partitions was necessary to capture the actual building response.

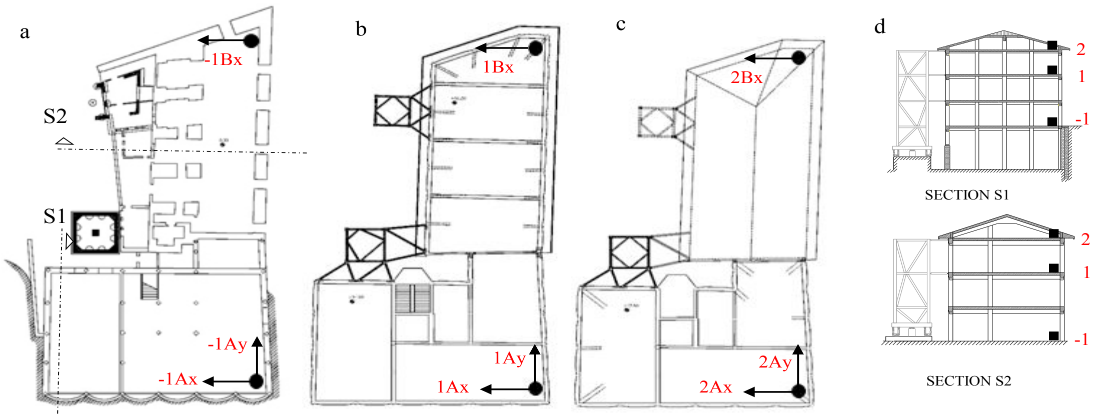

Fig. 4. Sensor positions: a) foundation level; b) 1st floor c) 2nd floor; d) Elevation view

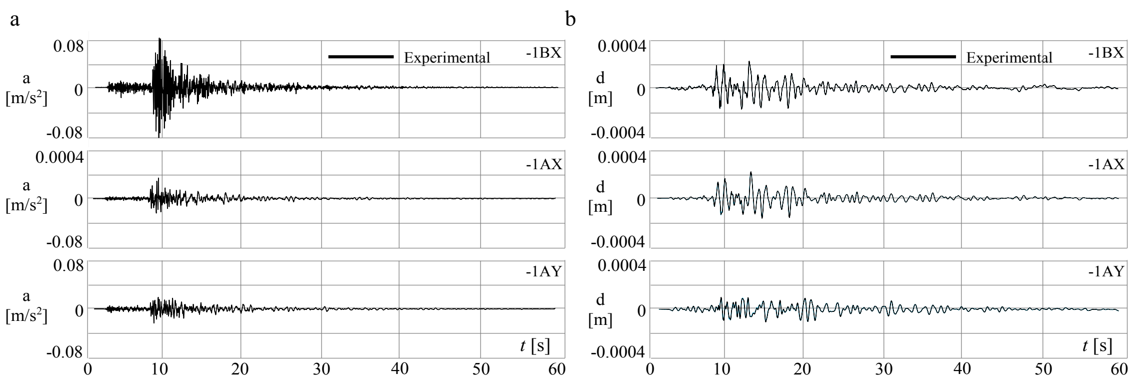

Fig. 5. a) Recorded earthquake in basement; b) Measured and analytical displacements in basement

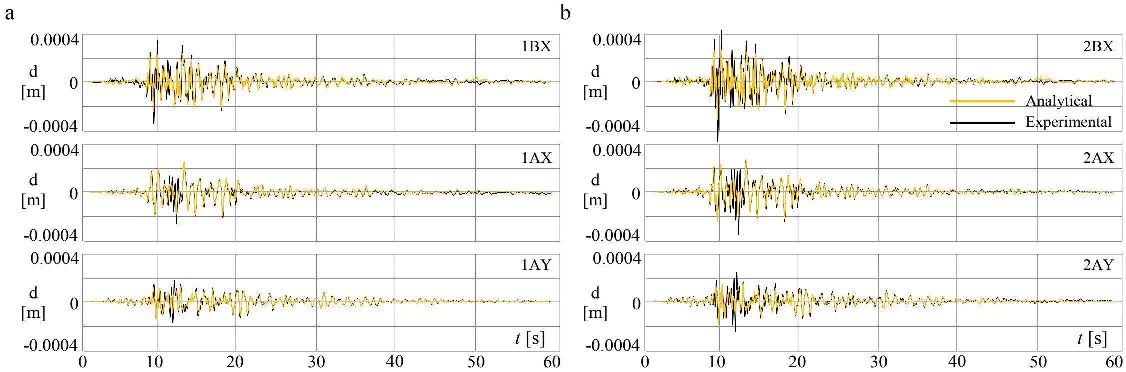

Fig. 6. a) Measured and analytical displacements in the 2nd floor; b) Measured and analytical displacements in the 1st floor

Numerical results, relevant to the compliant base model, are in good agreement with the experimental ones. In particular, the percentage errors in terms of Root Mean Square (RMS) of the numerical results with respect to the registered ones are about 17% and 10% for the second and first floor, respectively. However, some peak values of the measured acceleration, and the relevant displacement, are 50% greater than the theoretical ones; these discrepancies are likely due to the fact that the theoretical response of the building is evaluated with the model calibrated on the dynamic characteristics of the building experimentally obtained before the earthquake of 24th of August 2016. It is worth noting that the measured response is instead relevant to the building having reported (after the earthquake) moderate damages to light internal partitions.

6. Conclusions

The dynamic behaviour of an instrumented school building in Camerino, retrofitted by means of an innovative seismic protection system and subjected to some moderate earthquakes belonging to the after-shock sequence of the 2016 Central Italy earthquake, has been presented. The numerical model of the building, developed and calibrated starting form results of ambient vibration tests performed before the seismic event of August 2016, is used to interpret the registered structural response. Comparisons of numerical and registered data reveal the effectiveness of the developed numerical model in predicting the actual response of the structure and confirm the usefulness of ambient vibration tests in the framework of the seismic retrofit of structures for the definition of design numerical models, representative of the actual structural behaviour.

Acknowledgements

The authors thank Dr. Jakub Braja for his important contribution in the finite element model calibration and in performing the numerical simulations.

7. References

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