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Monitoring of the Modal Properties of a RC School Building During the 2016 Central Italy Seismic Swarm

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ABSTRACT

This paper presents results from the dynamic monitoring of a reinforced concrete school building located in Camerino (central Italy) during the seismic swarm following the first main shock of the Central Italy earthquake occurred in August 2016. After the main shock of August 24th, ambient vibration tests were executed on the building to identify its modal dynamic behaviour, which was assumed as benchmark to study changes in the structural response because of the subsequent events. A three days dynamic monitoring was then performed with the aim of investigating changes in the dynamic behaviour of the building subjected to ambient and seismic excitations. A procedure to identify the non-linear response of the structure subjected to seismic events is presented, starting from an optimization methodology that permits the identification of the structure dynamics within time windows in which the building dynamic behaviour can be considered linear time-invariant. Data show the variability of the modal properties, resonance frequencies and damping ratios of the building, with the earthquake intensity.

Author Keywords: *Structural Health Monitoring; Infilled RC frame building; Ambient vibrations; Seismic monitoring; Earthquake swarm; Dynamic system identification; Time-varying systems.*

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1. INTRODUCTION

In recent years, there has been a gradual increase in the attention concerning the usefulness and advantages offered by the permanent structural monitoring of civil engineering structures. Besides the possibility to monitor the structural health with time, that may change due endogenous (e.g. material ageing, material deterioration) and exogenous (ambient or anthropic actions) causes, a permanent monitoring system can capture the structural behaviour due to infrequent or rare events such as earthquakes. Recording the dynamic response of the structures during seismic events offers significant benefits: on one hand, it provides useful information for the damage identification and the post-earthquake emergency management (especially in the case of strategic buildings); on the other hand, it allows the reduction of seismic risk through the decrease of uncertainties related to both the hazard estimation (e.g. unexpected frequency contents of seismic events, unpredicted soil-structure interaction detrimental effects) and the structural vulnerability (e.g. unexpected damage or structural performance). The reduction of uncertainties about vulnerability can derive from the comparison of the real structural response for low-medium intensity events with the expected numerical one (e.g. obtained through the design model), while the reduction of uncertainties related to the hazard presupposes the monitoring of the free-field (nearby the building) and of the soil-foundation system.

With reference to a specific structural typology, the availability of simple and direct relationships between the variation of modal parameters (with respect to the undamaged condition), and the presumed damage levels, are of the utmost importance for the practical utility of data collected by monitoring systems (Ceravolo et al. 2016), especially in the case of medium-high seismic intensities. However, the definition of these relationships presents problems related to the intrinsic variability of the modal parameters of the structure with respect to both the intensity of the excitation and the environmental conditions.

As for low-medium intensity earthquakes, an interesting aspect emerging from the continuous monitoring of buildings is the well-known variability of the dynamic properties of the structures during shacking in absence of damage; in this sense, the wandering of the building modal parameters can be attributed both to the non-linearity of the response (Clinton et al. 2006) and to dependence on the environmental conditions (Rainieri et al. 2019a,b; Regni et al. 2018; Saisi et al. 2015). The aforementioned aspects make the vibration-based damage identification a non-trivial problem and several works can be found in literature addressing this issue for historic masonry buildings (Cavalagli et al. 2018; Ubertini et al. 2018; Gentile et al. 2016). On the contrary, few works refer to reinforced concrete (RC) structures; with specific regard to the frequency variation of RC buildings during strong seismic events, the work of Calvi et al. (2006) present an interesting state of the art of the problem. More recently, the variation in modal parameters of civil structures subjected to seismic actions has been the subject of several studies, including Ditommaso et al. (2012), Ghahari et al. (2013), Ceravolo et al. (2017), Hu and Xu (2019) and O'Reilly et al. (2019). However, considering the multiplicity of variables that may affect the structural response (e.g. construction typologies, non-structural elements) the number of case studies analysed and the scientific results available in literature are not enough to define a consolidate knowledge of the problem, especially with respect to RC structures.

With the aim of providing a contribution in this research field, this paper presents the dynamic investigation of a RC school building located in Camerino (central Italy) during the seismic swarm following the first main shock of the Central Italy earthquake, occurred in August 24th, 2016 of Richter Magnitude (ML) 6.0. A methodology to identify the non-linear response of the building subjected to seismic events is presented, starting from an optimization procedure that permits the identification of the building dynamics within time windows in which the building behaviour can be considered linear time-invariant. Firstly, the case study and its seismic retrofit carried out in 2013 are described, and details about the ambient vibration test carried out after the main shock of August are provided (e.g. measurement chain and sensor configurations). Results of the test, performed on August 27th, 2016, are presented in terms of modal parameters and are subsequently interpreted through a numerical refined finite element (f.e.) model of the building, in order to consolidate the reliability of data that will be used as benchmark to investigate changes in the dynamic response due to the seismic swarm. The permanent monitoring system installed for three days after the main shock is then described. The system foresees an accelerometer array able to measure continuously both the seismic excitation at the structure base, and the building response, due to both ambient vibrations and earthquakes. During the monitoring period many medium-low intensity earthquakes occurred and, although the building did not suffer significant damage, the modal parameters of the building, identified with the proposed methodology, underwent significant variations depending on the seismic input level. These variations are analysed and discussed, with attention to the relationships between the modal parameters and the seismic event intensity.

2. THE CASE STUDY

2.1 Building description

The investigated case study is a RC frame building built around the year 1960 and hosting the Costanza da Varano high school in the historical town centre of Camerino. The town was overall severely damaged by the Central Italy earthquake in 2016, especially by the main shocks of October, 26th and October, 30th of ML 5.9 and 6.1, respectively. A significant number of masonry building (e.g. the church of Santa Maria in Via, the cathedral of Santa Maria Annunziata, Palazzo Ducale) and RC buildings (e.g. the old courthouse of the town), were damaged and also some partial collapses occurred.

The building, characterised by an L-shaped plan, is divided into two main blocks as shown in Fig. 1a: block A, having 4 storeys (1 underground and 3 above ground) and plan dimensions around 25 x 19 m; and block B, having 3 storeys above ground and plan dimensions of about 13 x 27 m. Blocks are separated by a 2 cm wide joint in correspondence of the structural elements, whereas the non-structural components (screeds, floorings and infill walls) are continuous through the joints. Moreover, block A is further divided into two parts (A1 and A2) by means of another structural joint. The block A is directly founded on the sandstone rock by means of RC plinths, while block B is built over the ancient masonry ruins belonging to the S. Elisabetta Convent. All columns have square cross-sections (40 x 40 cm) and are rotated of 45° with respect to the frame plane for aesthetical reasons. Beams located at the building perimeter are linearly tapered with a cross section of about 30 x 80 cm at the beam-to-column joints and 30 x 40 cm at midspan, whereas the internal beams have uniform rectangular cross section with different dimensions depending on the beam position. It is worth noting that, due to the cross-section dimensions of structural components (columns and beams), frames tend to develop a shear-type horizontal behaviour. All the beam and hollow block floors have a thickness of 24 cm. Internal partitions are constituted with light infill masonry walls, while external ones are 1.2 m high double brick walls, which leave space for large windows (Fig. 1b).

In 2013 a seismic retrofit was carried out in order to improve the building seismic performance, which suffers of intrinsic vulnerabilities deriving from the structural element dimensions (e.g. strong beam, weak columns). The main blocks were structurally connected each other with thick steel plates anchored to the RC elements in correspondence of the structural joints. Then, two external steel truss towers (Fig. 1a,b), called *dissipative towers* (Balducci 2005a), were built and rigidly connected with the building at the floor levels by means of steel braces anchored to the external beams.

Tower Ta is the tallest one (14.5 m) and is connected to both the block A (at the upper three floors) and the block B (at the upper two floors); tower Tb (9.3 m high) is connected only to the block B (at the upper two floors). Each tower is erected on a RC thick base plate that is centrally pinned to the foundation plate by means of a spherical support. Eight and four viscous dampers for tower Ta and Tb, respectively, are located in vertical position between the base and foundation plates (one or two devices per vertex), so that the base plate rigid rotations, due to the horizontal building displacements, activate simultaneously all the devices. Articulated quadrangles are adopted to amplify the device displacements through leverage systems. Towers are founded on piles and micropiles to transfer both compression and tension forces during earthquakes arising from the viscous forces transferred by the devices acting at the plate vertexes. More details about the seismic retrofit of the school building can be found in Balducci et al. (2015) and in Gara et al. (2020).

Figure 1 is approximately here

2.2 Identification of the building dynamics

Some dynamic tests were performed in the recent past in order to identify the building dynamic behaviour during different retrofitting work stages. More specifically, ambient vibration tests (AVTs) were carried out before and after the seismic retrofit to identify the modal parameters characterizing the building in these two important phases. The purpose was twofold: first, the preliminary identification made it possible to obtain a calibrated f.e. model of the structure for the retrofit design and, second, the investigation after the seismic retrofit made it possible to verify that the modal parameters of the structure correspond to those predicted by the developed f.e. model, assessing its reliability and usefulness also in a monitoring process. A broader description of the performed tests and obtained results can be found in Gara et al. (2020). The building experienced the Amatrice earthquake of August 24th, 2016, which was perceived strongly in Camerino; however, the building suffered negligible damage mainly consisting in light internal partition cracks, which were immediately repaired to guarantee the building occupancy and the regular start of the school in September. After this seismic sequence, a new AVT was performed on August 27th, 2016, in order to assess the health status of the building and to obtain results to be used as benchmark for outcomes of the following monitoring.

The instrumentation adopted to perform the dynamic test (Fig. 2a) consisted in low-noise uniaxial piezoelectric accelerometers with ceramic flexural ICP, model PCB 393B31, sensitivity of 10 V/g, broadband resolution of 1 µg rms, measurement range of 0.5 g pk and frequency range between 0.1 and 200 Hz. Sensors were connected by means of coaxial cables with BNC connectors to four 4-channels dynamic signal acquisition modules NI-9234 characterized by measurement range of 0.5 g pk, ADC resolution of 24 bits, signal ranges of ± 5 V and sample rate of 51.2 kS/s/ch, mounted on a 8-slot USB chassis NI cDAQ 9178. A laptop equipped with a dedicated software developed in Labview environment was adopted to acquire signals and to store the data. 30 minutes long records sampled at a rate of 2048 Hz were acquired during the tests. Three accelerometers per floor were used: two measuring in X direction and one in Y direction (Fig. 2b). Furthermore, two sensors for each dissipative tower were employed (Ta_1 and Ta_2 for tower Ta, and Tb_1 and Tb_2 for tower Tb), placed on the RC base plate (the one over the spherical hinge) and measuring in vertical direction (shown as black arrows in the sensors layout of Fig. 2b). The adopted measurement configuration makes it possible to investigate the whole building dynamic behavior considering also the tower modal displacements relevant to each building vibration mode (for the reconstruction of mode shapes, towers are assumed to be rigid, in good consistency with the design assumptions).

At first, usual pre-processing signal procedures were performed, consisting in correction of signal spurious trends using a third-degree polynomial function, low pass filtering of the analogic signal above the Nyquist frequency with cut-off frequency of 25.6 Hz (to eliminate the contribution of high frequencies and avoid aliasing phenomena) and down-sampling of the signal at 51.2 Hz in order to limit the amount of data to be managed. Then, the modal parameters of the building were identified through Operational Modal Analysis (OMA). The ambient excitation is unknown and is assumed to have a flat spectrum such as a white noise; therefore, the modal parameters were identified through the Covariance-driven Stochastic Subspace Identification (SSI-COV) output-only technique (Van Overschee and De Moor 1996), which works in time domain. In Fig. 3 results of the dynamic characterization are summarised; in detail, Fig. 3a shows the first three mode shapes, drawn on the basis of the rigid floor assumption and considering the dissipative towers as non-deformable systems. The first vibration mode is a roto-translational mode in the X transverse direction with higher modal displacements in proximity of the block A; the second one is mainly a rotational mode, while the third one is a roto-translational mode in the Y longitudinal direction. The mode shapes are almost orthogonal to each other, as can be observed from the Auto-Modal Assurance Criterion (Auto-MAC) matrix reported in Fig. 3b, where the off-diagonal terms have very low values.

Figure 2 is approximately here

Low values of off-diagonal terms also demonstrate that the considered degrees of freedom are enough to avoid the spatial aliasing problem in describing the first building mode shapes (i.e. the rigid floor assumption is acceptable and the use of three sensors per floor is justified). Moreover, it is possible to note that the identified modes are real structural modes since the modal complexity is negligible. This is evident from the Argand diagrams of Fig. 3c, and from the very low values of Mode Complexity Factor (MCF) reported in the summary table of Fig. 3d.

Results of the AVT are interpreted through a refined f.e. model of the whole building, developed by means of a commercial software (Fig. 4a). Both beams and columns are modelled with elastic frame elements, while shell elements are used to model the floors and stair slabs in order to both account for their in-plane and out-of plane deformability. The ancient masonry walls at the base of block B and the external and internal infill masonry walls were also modelled through shell elements. The modelling is based on available structural drawings of the building and in-situ measurements as well as destructive and non-destructive tests on the structural materials; in detail, the dynamic concrete elastic modulus, obtained by increasing the static one by about 20% (Lydon and Balendran 1986), is assumed to be 30720 MPa while for the density the value of 2.5 t/m^3 is considered. Concerning the ancient masonry, the external and internal walls, dynamic elastic moduli of 4032, 5500 and 3850 MPa are considered, respectively, based on indications provided by the Italian Standards (Circolare 21.01.2019), depending on the masonry typology. For the previous walls, densities of 2.2, 2.0 and 1.2 t/m^3 are adopted, respectively. In order to simulate the localised deformability due to structural joints between the building blocks sewed through steel plates, elastic links are adopted between the modelled frame elements having stiffness calibrated starting from the AVT results. The base joints are fixed and the foundations are not modelled, since the building is founded on cement sandstone, but the contribution of the soil surrounding the ancient masonry on one side of the building is taken into account through springs having stiffnesses obtained assuming a subgrade reaction value of 80000 kN/m^3, within the range suggested by Bowles (1996) for a medium-dense sand. Further details concerning material properties, the overall modelling and the model updating process can be found in Gara et al. (2020).

As for the dissipative towers, the braced steel frames are schematized with beam elements, while the RC base plate is modelled with shell elements; the base plate, as well as the dissipative devices, are pinned to the ground.

Figure 3 is approximately here

An eigenvector analysis is performed to get the numerical modal parameters of the building, which are in very good agreement with the corresponding experimental ones (Fig. 4b, c, d) that are determined assuming the in-plane rigidity of floors, consistently with the available number of measuring points. It is worth observing that the good matching of experimental and numerical results supports validity of the hypothesis of the in-plane rigidity of floors, which was used to derive the experimental mode shapes. This is evident by observing the Modal Assurance Criterion (MAC) matrix between numerical and experimental mode shapes (Fig. 4d) where the MAC indexes along the diagonal entries are close to 100%. The assumption of rigid behaviour adopted for the reconstruction of the tower mode shapes is also validated.

Figure 4 is approximately here

3. THE CONTINUOUS MONITORING SYSTEM

3.1 Sensor configuration and seismic events

After the preliminary AVT performed on August 27th, 2016, a continuous dynamic monitoring system was installed on the building and left operative for three days, with the aim of monitoring the dynamic behaviour of the building during the seismic swarm following the main shock. The monitoring system was composed by the same instrumentation adopted to perform the benchmark AVT previously described. In this case, thirteen low-noise uniaxial piezoelectric accelerometers were adopted with the layout shown in Fig. 5: three sensors were positioned at the base floor (-1st floor) to measure the seismic input in the two horizontal orthogonal directions, six accelerometers on the 1st and 2nd floors (three for each one) to measure the structural response, and four accelerometers on the tower base plates (two for each plate) to capture the movements of the towers. Although signals registered at the building base do not correspond to the free-field seismic motion at the building location, due to kinematic and inertial soil-structure interaction effects (Gazetas et al., 2006; Capatti et al., 2017), in the sequel they will be referred to as seismic input, for the sake of simplicity. Both the input (seismic input) and the output (building response) were recorded using a 2048 Hz sampling rate.

During the whole monitoring period the data were acquired continuously allowing the monitoring of both the dynamic behaviour of the building subjected to environmental vibrations (i.e. between two subsequent events of the seismic swarm) and its response to seismic events. Records of the building response due to ambient vibrations were divided into 20 minutes recordings; for each time history an output-only identification was carried out through the SSI-COV method, the same adopted for the benchmark AVT. The tracking of modal parameters obtained from data in absence of earthquake provides useful information to investigate the evolution of possible structural and non-structural damage occurred during the seismic sequence, as well as to investigate modal property changes due to environmental effects. Records of the building response due to seismic events have been isolated and used to identify the building dynamics during their occurrence, as shown in the sequel.

Figure 5 is approximately here

During the three days of monitoring many seismic events occurred from low to medium intensity and only those with ML greater than 2.6 were selected for the subsequent analyses. Table 1 reports the twenty-five considered seismic events with their occurrence data and time, intensity (ML), hypocentre depth and epicentre distance from the investigated building; events are sorted by decreasing intensity and the strongest one, occurred on August 28th, 2016, at 5:55 p.m., is characterized by 4.4 ML and epicentre distance of about 37 km from the school. In Fig. 6 the positions of the seismic event epicentre are depicted together with the main features of the most relevant ones (first 4 events of Table 1).

Table 1 is approximately here

Figure 6 is approximately here

Fig. 7 shows accelerations, velocities and displacements recorded by three sensors (2Ay, -1Ay and Ta_1) of the monitoring system during the most intense event (with 4.4 ML). It is worth observing that events of the seismic swarm induced overall low accelerations, velocities and displacements to the structure. In detail, the latter are of the order of 10^{-1} mm at the second floor and of 10^{-2} mm at the base of the dissipative towers, at the level of the viscous dampers.

Figure 7 is approximately here

3.2 Dynamic characterization of the building subjected to earthquakes: brief overview of the subspace identification methods and proposed methodology

Subspace identification methods have proven to be reliable and robust approaches for the dynamic characterization of complex multi-input multi-output (MIMO) dynamic systems with close eigen-frequencies and have been successfully used for several years also in the field of civil engineering. In particular, two of the most popular algorithms used for combined MIMO systems identification (deterministic and stochastic) are the Multivariable Output Error State Space (MOESP) (Verhaegen 1994) and the Numerical algorithm for Subspace State Space System IDentification (N4SID) (Van Overschee and De Moor 1996). Skolnik et al. (2006) adopted the N4SID algorithm to identify the dynamics of a 17-story steel moment resisting frame, the UCLA Louis Factor building, during low-amplitude earthquakes; Ceravolo et al. (2016) used the same algorithm to investigate the dynamic response of three buildings subjected to the seismic swarm occurred in Lunigiana-Garfagnana starting from June 21st, 2013; Illescas et al. (2019) used both the N4SID and the MOESP for the structural health monitoring of an elevated railroad segment of Mexico City Metro Line 12, while Boroschek et al. (2013) implemented the MOESP algorithm to identify the dynamic of a building subjected to the 2010 Gigantic Chile Earthquake.

As known, the first step to use a subspace identification methodology, is to represent the structural dynamics through a state space system model, which is described by the following set of equations, including a state equation (Eq. 1a) and an output equation (Eq. 1b) (process form):

$$\mathbf{x}_{k+1} = \mathbf{A}\mathbf{x}_k + \mathbf{B}\mathbf{u}_k + \mathbf{w}_k \tag{1a}$$

$$\mathbf{y}_k = \mathbf{C}\mathbf{x}_k + \mathbf{D}\mathbf{u}_k + \mathbf{v}_k \tag{1b}$$

where $\mathbf{u}_k \in \mathbb{R}^m$ and $\mathbf{y}_k \in \mathbb{R}^l$ denotes the input and output signals, respectively, at a certain time *k*, while $\mathbf{x}_k \in \mathbb{R}^n$ is the state vector. In addition, $\mathbf{A} \in \mathbb{R}^{n\times n}$ is the dynamical system matrix, $\mathbf{B} \in \mathbb{R}^{n\times m}$ is the input matrix that describes how the deterministic inputs influence the next state, $\mathbf{C} \in \mathbb{R}^{l\times n}$ is the output matrix that characterizes how the internal state influence the outputs and $\mathbf{D} \in \mathbb{R}^{l\times m}$ is the direct transition matrix. For a linear time-invariant system above matrices are constant. Furthermore, $\mathbf{w}_k \in \mathbb{R}^n$ and $\mathbf{v}_k \in \mathbb{R}^l$ are unmeasurable vector signals, which are assumed to be normally distributed, zero mean, white noise signals for which:

$$E\left[\begin{pmatrix}\mathbf{w}_p \\ \mathbf{v}_p\end{pmatrix}\begin{pmatrix}\mathbf{w}_q^T & \mathbf{v}_q^T\end{pmatrix}\right] = \begin{pmatrix}\mathbf{Q} & \mathbf{S} \\ \mathbf{S}^T & \mathbf{R}\end{pmatrix}\delta_{pq} \geq 0 \tag{2}$$

where *E* is the expected value operator and δ_{pq} is the Kronecker delta. Finally, $\mathbf{Q} \in \mathbb{R}^{n\times n}$, $\mathbf{S} \in \mathbb{R}^{n\times l}$ and $\mathbf{R} \in \mathbb{R}^{l\times l}$ are matrices of suitable dimensions. The mathematical problem, which is solved through the N4SID algorithms, is that of identifying matrices **A**, **B**, **C**, **D**, **Q**, **R** and **S** given input and output measurements. It is well known that Eq. 1 can be also expressed as (innovation form):

$$\mathbf{x}_{k+1} = \mathbf{A}\mathbf{x}_k + \mathbf{B}\mathbf{u}_k + \mathbf{K}\mathbf{e}_k \tag{3a}$$

$$\mathbf{y}_k = \mathbf{C}\mathbf{x}_k + \mathbf{D}\mathbf{u}_k + \mathbf{e}_k \tag{3b}$$

where **K** is the steady state Kalman gain while $\mathbf{e}_k$ is a white noise, independent of past input and output data. Finally, the system can be also expressed in the predictor form:

$$\mathbf{x}_{k+1} = \mathbf{A}_k\mathbf{x}_k + \mathbf{B}_k\mathbf{z}_k \tag{4a}$$

$$\mathbf{y}_k = \mathbf{C}\mathbf{x}_k + \mathbf{D}\mathbf{u}_k + \mathbf{e}_k \tag{4b}$$

where

$$\mathbf{z}_k = [\mathbf{u}_k^T \quad \mathbf{y}_k^T]^T \tag{5a}$$

$$\mathbf{A}_k = \mathbf{A} - \mathbf{K}\mathbf{C} \tag{5b}$$

$$\mathbf{B}_k = [\mathbf{B} - \mathbf{K}\mathbf{D} \quad \mathbf{K}] \tag{5c}$$

Similarly to Eq. 1, Eq. 3 and Eq. 4 are able to represent the input and output data; for instance, the MOESP algorithm uses the innovation form (Eq. 3) while other approaches use the predictor form (Chiuso and Picci 2005).

Considering the case study discussed before, and taking into account experiences form the literature, the main problem to face in the application of above algorithms is the nonlinear nature of the building response for increasing amplitudes of the accelerations to which the structure is subjected. The nonlinear behaviour is evident observing diagrams of Fig. 8, where Short Time Fourier Transforms (STFTs) of the signals recorded in the measuring points 1Ax, 1Ay, 1Bx, 2Ax, 2Ay and 2Bx during the main seismic event (4.4 ML) are reported.

It can be observed that, during the strong motion, the dynamics of the building varies significantly, with an evident reduction of the frequency content of the registered signal with the increase of the acceleration amplitude. However, at the end of the strong motion, the resonance frequencies tend to attain the same values of those governing the initial part of the time series. Thus, the system dynamics is clearly time-varying and the state space system matrices change over the time k. There are several works in literature dealing with the identification of time-varying systems through subspace methods. Tamariz et al. (2005) developed an iterative state-space identification algorithm for discrete time-variant systems, based on MOESP type subspace methods and called MOESP-VAR, following the basic idea that a linear operator can be described as a composition of local linear transformations. Further interesting approaches can be found in Robles et al. (2018), where a version of the N4SID algorithm for the identification of multivariable linear time-variant systems, named N4SID-VAR, is developed, and in Loh and Chen (2017), where several methods are used to keep track of modal parameters from structural seismic response data.

Figure 8 is approximately here

In this work, an iterative procedure is proposed, consisting in tracking the evolution of the dynamic parameters of the system starting from the identification made on signal windows within which the dynamic behaviour can be assimilated to that of a linear time-invariant system. An extended description of the procedure is herein reported using one of the twenty-five seismic events recorded during the monitoring (the strongest one with 4.4 ML), while the overall results of the monitoring will be presented in the next section. The proposed procedure aims to optimize the number of samples, and therefore the length of the windows, in which the system dynamics can be described as a linear time-invariant process. The length for the first iteration is deduced from a preliminary time frequency analysis and a short window is initially selected. For the generic window length, the identification is carried out through a subspace identification methodology, and the obtained dynamic model is used to predict, starting from the recorded seismic input, the analytical response of the building which is compared with the registered one. Thereafter, the window length is adjusted until the system identified in the initial window is able to accurately predict the structure response to the event, namely the length of the window is adjusted until the model accurately predicts the experimental response. The steps of the optimization procedure are summarized in the flow chart reported in Fig. 9. At the end of the process, a set of optimal time windows are determined in which the overall signal can be divided; the response of the system in each window can be considered to be time-invariant and the system can be identified through a subspace identification method. The proposed approach allows tracking the evolution of the modal parameters of the system during the shaking.

The identification within each window has been made through the "robust combined algorithm" proposed by Van Overschee and De Moor in (1996). This algorithm consists in a Singular Value Decomposition (SVD) of a weighted projection matrix for the determination of the model order. Then, the state space matrices **A**, **B**, **C** and **D**, and the corresponding covariance matrices **Q**, **S** and **R**, are determined by solving a set of linear equation, according to the N4SID algorithm. The system inputs are two time-histories recorded at the building foundation level (point -1Ax and -1Ay), while, as outputs, the eleven time-histories recorded by all the other sensors, are adopted. Both the inputs and the outputs were detrended in order to remove any slope and mean offset and filtered through a band-pass filter between 1 e 10 Hz with the aim of considering only the contribution of the building dynamics.

The accuracy of the identified model in reproducing the response of the building is assessed using the comparison metrics proposed by Kavrakov et al. (2020), which consider different signal properties. In detail, metrics are constructed using the following exponential function:

$$M(u_e, u_a) = \exp\left(-\lambda |A(u_e, u_a)|\right) \qquad (6)$$

so that results vary between 0 and 1. In Eq. 6, u_e and u_a are the experimental and analytical response that have to be compared, λ is the metric parameter (assumed to be equal to 1) and A is suitably constructed to account for a particular property of the signals.

In particular, the phase M_φ, the peak M_p and the root mean square M_{rms} are obtained considering the following exponents:

$$A_{\varphi} = \frac{t_{lag}}{T_c} \qquad t_{lag} = \arg max_t u_e(t) * u_a(t) \tag{7a}$$

$$A_p = \frac{max_t|u_e(t)| - max_t|u_a(t)|}{max_t|u_e(t)|} \tag{7b}$$

$$A_{rms} = \frac{\sqrt{\int_0^T [u_e(t)]^2 dt} - \sqrt{\int_0^T [u_a(t)]^2 dt}}{\sqrt{\int_0^T [u_e(t)]^2 dt}} \tag{7c}$$

The phase metric accounts for the mean phase discrepancy between signals, with respect to the reference time delay T_c; the latter coefficient depends on what is considered to be a large delay between the signals; in this case, the period of the first mode has been used. The peak metric M_p accounts for the difference in the maximum peak response, while the root mean square metric M_{rms} quantify discrepancies of signals with respect to their average quantities. Furthermore, to evaluate the signal differences in the time-frequency plane, two further metrics based on the wavelet transform have been used: the wavelet metric M_w, which allows studying the overall signal discrepancies in the time-frequency plane, and the frequency normalised wavelet metric M_{wf}, which allows to understand if these discrepancies are due to the signal amplitudes or frequency content. The relevant metric exponents are obtained with the following expressions:

$$A_w = \frac{\int_0^\infty \int_0^T \left| |W_{u_e}(a,t)| - |W_{u_a}(a,t)| \right| dtda}{\int_0^\infty \int_0^T |W_{u_e}(a,t)| dtda} \tag{8a}$$

$$A_{wf} = \int_0^T \frac{\int_0^\infty \left| \frac{|W_{u_e}(a,t)|}{max_a|W_{u_e}(a,t)|} - \frac{|W_{u_a}(a,t)|}{max_a|W_{u_a}(a,t)|} \right| da}{\int_0^\infty \frac{|W_{u_e}(a,t)|}{max_a|W_{u_e}(a,t)|} da} dt \tag{8b}$$

where $W_{u_i}(a,t)$ for $u_i(a,t)$ is obtained as

$$W_{u_i}(a,t) = \frac{1}{\sqrt{|a|}} \int_{-\infty}^{\infty} u_i(\tau) \psi\left(\frac{t-\tau}{a}\right) d\tau \tag{9}$$

in which a is the scale and ψ is the Morlet wavelet. In this work, the analytical response predicted within a window has been considered “accurate” if the metric values obtained from analytical and experimental signal comparison in each superstructure measurement point, are all greater than 0.8. In this case, the optimal length has been reached with a few iterations thanks to the nature of the recorded events which did not induce extreme variations of the dynamic system. However, the method can be improved by implementing machine learning procedures (e.g. Bayesian optimization, neural networks) that could automatically identify the number and the length of time windows.

Figure 9 is approximately here

Fig. 10 shows the results of the procedure carried out on the time history recorded in position 2Ay during the 4.4 ML earthquake. In detail, Fig.10a shows the time-frequency analysis of the signal from which the initial lengths of the windows are established, while Fig. 10b shows the acceleration time histories (i.e. measured from the monitoring system and predicted through the identified dynamic time invariant systems) and provides indication of the set of windows identified with the proposed approach. A more detailed comparison between the predicted and measured time histories for each window is depicted in Fig. 10e, where it is possible to observe that the predicted and measured responses are almost superimposed, demonstrating the effectiveness of the proposed methodology.

Figure 10 is approximately here

In Fig. 10c the resonance frequencies identified in each window for the first three vibration modes are reported. For all the vibration modes, resonance frequency values decrease during the strong motion, where the maximum accelerations occur, and then increase again at the end of the strong motion, returning to values very close to the initial ones; for the presented earthquake and the first vibration mode, the initial frequency value is 3.51 Hz, the lowest one identified during the strong motion is 2.85 Hz and the frequency achieved at the end of the signal is 3.39 Hz, very close to the first one. Also for damping ratios (Fig. 10d) a trend is clearly evident, with values increasing in correspondence of the maximum accelerations and then decreasing at the end of the earthquake, towards values close to those identified at the beginning; for the presented earthquake and the first vibration mode the initial damping ratio is 1.91%, the highest one is 5.34% and the value at the end of the signal is 2.24%. Table 2 summarizes the comparison metrics for all the monitoring points in which the output is measured, and for all signal windows; metrics relevant to signal 2Ay analysed in Fig. 10 are in bold. As can be observed, the identified systems accurately predict the structural response, since all the metric values are greater than 0.8 and, consequently, the system is assumed to be time-invariant within each window.

In Fig. 11 the first three mode shapes identified from each windowed signal are depicted, together with those identified from the OMA (AVTs) performed on acceleration measurements recorded before and after the considered seismic event (i.e. from the response of the building to ambient vibrations before and after the seismic shaking). Similarly to frequencies and damping ratios, also mode shapes evaluated before and after the seismic event are almost the same as can be deduced comparing the first and last rows of mode shapes (in dark grey) in Fig. 11. Differently, mode shapes identified from signals in each window through input-output technique are drawn in light grey since MCF values, which evolve during the shaking, indicate a non-negligible complexity of the identified modes. In any case, absolute values of the modal displacements of the monitored points are highlighted with red lines to provide an idea of the identified modal shape. The very low values of the MCF identified from the OMA procedure confirm that modes are almost real before and after the earthquake while the higher values obtained during the motion may be consequence of the in-plane floor compliance. Furthermore, it is interesting to note that translational modes tend to decouple from a torsional behaviour and, in correspondence of the highest accelerations, the first two modes are translational while the third is torsional. The latter phenomenon is probably due to the reduction of the contribution of non-structural elements (e.g. infill wall) in the dynamic response of the building.

Table 2 is approximately here

4. MONITORING RESULTS

The results obtained from the three days of monitoring in terms of evolution of the building resonance frequencies are summarized in Fig. 12. In detail, Fig. 12a refers to the results achieved from the ambient excitation data (i.e. from the recordings between two subsequent seismic events); the values of the resonance frequency for the first three vibration modes of the building are identified almost every twenty minutes; signal windows adopted for the OMA are characterised by a root mean square of the accelerations within the range $0.8 \cdot 10^{-5} \div 4.4 \cdot 10^{-5}$ m/s^2. The first frequency value for each mode is very close to that identified from the benchmark test described in Section 2, since both are obtained in the same day and almost at the same time (after the benchmark test the permanent monitoring system was immediately installed and made operative). Overall, the presence of a daily trend in the frequency data can be observed: the highest frequency values are identified during the day warmer hours, around 1:00 p.m., while the lowest ones during the night.

Figure 11 is approximately here

Figure 12 is approximately here

The weathering conditions during the monitoring days were registered in terms of maximum and minimum temperature: the weather was sunny or partially cloudy all days, with maximum temperatures around 27°C at mid-day and minimum temperatures of about 16°C during the night. The observed trend is clearly unrelated to the seismic sequence and the frequency value oscillations are due to the temperature effects, with frequency values increasing as the temperature increases. This phenomenon is known in literature, both for historic masonry buildings (Gentile et al. 2019) and for RC buildings (Arezzo et al. 2019; Rainieri et al. 2019; Regni et al. 2018).

The resonance frequencies identified through the proposed methodology and considering the twenty-five seismic events are superimposed to data obtained from AVTs in Fig. 12b and reported with triangles whose dimensions increase with the event Peak Ground Acceleration (PGA). For each earthquake only the frequency values obtained considering the signal window including the PGA are herein considered. It can be clearly observed that frequencies identified during the strong motion are sensibly lower than the benchmark values and the values obtained from the OMA performed on ambient acceleration measurements made before and after the shaking. In addition, the frequency reduction increases with the earthquake PGA; this can be deduced from Fig. 12c where the PGA of events are represented with circles whose dimensions increase with the event PGA. Obviously, in correspondence of the strongest earthquake (in the afternoon of August, 28th) the lowest frequency values are attained (dashed line). Moreover, the frequency values identified at the monitoring end and based on ambient vibration data, are almost the same obtained at the beginning of the monitoring. Thus, it can be stated that the building dynamics at very low intensity actions, such those produced by ambient excitation, has not changed after the seismic swarm observed in the monitoring days and the frequency values reduction during the strong motions can be attributed to nonlinear phenomena of secondary importance (e.g. friction phenomena due to non-structural members, opening and re-closing of infill small cracks) rather than to structural or non-structural damage. Indeed, when the permanent monitoring system was removed due to logistic problems related to the beginning of the school, the building did not present evident damage that could be attributed to the registered seismic events.

In order to better investigate the relationships between the identified modal parameters and the features of the occurred seismic events, some correlations are determined and addressed hereafter. In Fig. 13a, the correlations between the modal parameters (the first three resonance frequencies and the relevant damping ratios) identified during the single events and the PGA of the corresponding event are reported in semi-logarithmic graphs. Data are interpolated with logarithmic functions and the relevant coefficient of determination (R^2) are reported in the graph. It is clear that resonance frequencies decrease with the increasing of the PGA while the damping ratios increase with the increasing of PGA. Similarly, Fig. 13b shows the correlations between the modal parameters and the maximum displacement (d_{max}), obtained from the accelerometers at the top floor, during the strong motion of the seismic events. Finally, Fig. 13c refers to correlations between the modal parameters and the maximum acceleration (a_{max}) measured at the top floor. It can be concluded that all the selected intensity measures are well correlated with the modal parameters, presenting very similar correlation coefficients.

Figure 13 is approximately here

Fig. 14 shows the correlation between the frequency values and the relevant damping ratios, which is quite well interpreted through a linear trend. Overall, the increase of the damping ratios is consistent with the reduction of the frequencies and the interpretation provided above, which attributes the resonance frequency reduction to the development of secondary nonlinear phenomena, such as light cracking and frictions due to interactions between structural and non-structural members. Indeed, values of damping ratios, achieving a maximum around 5% in correspondence of the strongest events, suggest that the dissipative phenomena cannot be attributed to the dissipative system installed for the building seismic retrofit. This conclusion is supported by the order of magnitude of velocities and most important displacements registered at the base of the towers, which are not deemed to be sufficiently high to activate the dissipative mechanisms (Fig. 7).

Figure 14 is approximately here

5. CONCLUSIONS

The dynamic monitoring of the Costanza da Varano high school building subjected to part of the seismic swarm that followed the Amatrice earthquake in central Italy on August 2016 has been presented in this paper. The building was instrumented with 13 piezoelectric low-noise accelerometers recording in continuous for three days in which many seismic events occurred, ranging from Richter Magnitude 2.6 to 4.4. Data acquired by the permanent monitoring system permitted to capture the dynamic behaviour of the building subjected to both ambient and seismic excitations. The dynamic characterization of the school was performed considering both the non-seismic and seismic registrations. As for the latter, a methodology to identify the modal parameters starting from the input (seismic excitation) and output (building response) records has been proposed. The methodology, which consists in the identification of the structure dynamics within time windows in which the response is linear time-invariant, is applied for the identification of the building modal parameters during the twenty-five strong motions with Richter Magnitude greater than 2.6 which occurred during the monitoring. The proposed algorithm uses comparison metrics between the experimental signals recorded during the seismic events and the corresponding analytical signals predicted by the identified dynamic models to define time windows characterised by time-invariant response of the building. The procedure permits the tracking of the evolution of the dynamic structural properties with time, and precisely with the increase of the accelerations.

By focusing on the dynamic properties of the system in the neighbourhood of the highest accelerations of each event, correlations between the first resonance frequencies of the building and the peak ground accelerations are determined. It was found that frequency values decrease as the seismic intensity increases while damping ratios increase. At the end of the shaking, frequency values close to the benchmark ones obtained before the beginning of the monitoring are obtained. Data have been interpreted through the development of secondary nonlinear effects such as frictions between structural and non-structural members or light opening and re-closing of infill cracks, which reduce the building stiffness and increase its dissipative capabilities. The reduction of the resonance frequency values in correspondence of the seismic events is not permanent and, after the shock, the building returns at the state identified at the beginning of the monitoring.

Finally, from the observation of the resonance frequencies identified from ambient vibration measurements, a clear daily fluctuation was observed with values that increase during the day and decrease during the night, likely due to temperature effects on the building.

6. DATA AVAILABILITY STATEMENT

Some or all data or codes that support the findings of this study are available from the corresponding author upon reasonable request.

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TABLE 1. Considered seismic events during the three days of monitoring.

Event n.	Intensity [ML]	Hypocentre [km]	Epicentre [km]	Date (2016)	Time	Website
1	4.4	9	36.93	Aug. 28th	5:55 pm	http://cnt.rm.ingv.it/event/7343701
2	3.8	9	35.40	Aug. 28th	6:42 pm	http://cnt.rm.ingv.it/event/7345471
3	3.7	9	61.97	Aug. 28th	3:07 pm	http://cnt.rm.ingv.it/event/7339051
4	3.6	10	43.47	Aug. 29th	8:20 am	http://cnt.rm.ingv.it/event/7370871
5	3.6	12	40.30	Aug. 28th	5:37 pm	http://cnt.rm.ingv.it/event/7343051
6	3.5	10	42.26	Aug. 29th	3:44 am	http://cnt.rm.ingv.it/event/7363391
7	3.4	11	62.03	Aug. 27th	11:31 pm	http://cnt.rm.ingv.it/event/7307161
8	3.4	11	46.88	Aug. 28th	8:37 am	http://cnt.rm.ingv.it/event/7326791
9	3.2	8	59.20	Aug. 28th	11:18 am	http://cnt.rm.ingv.it/event/7332041
10	3.1	8	45.02	Aug. 28th	10:22 pm	http://cnt.rm.ingv.it/event/7353481
11	3.1	9	62.03	Aug. 28th	7:16 am	http://cnt.rm.ingv.it/event/7323941
12	3.0	7	39.83	Aug. 28th	9:59 am	http://cnt.rm.ingv.it/event/7329641
13	3.0	10	39.42	Aug. 28th	12:25 pm	http://cnt.rm.ingv.it/event/7334431
14	2.9	8	59.75	Aug. 28th	8:13 am	http://cnt.rm.ingv.it/event/7325951
15	2.9	10	60.11	Aug. 28th	1:53 am	http://cnt.rm.ingv.it/event/7312881
16	2.8	11	46.83	Aug. 28th	6:25 pm	http://cnt.rm.ingv.it/event/7344771
17	2.8	10	57.24	Aug. 27th	11:26 pm	http://cnt.rm.ingv.it/event/7306911
18	2.8	10	37.31	Aug. 27th	7:50 pm	http://cnt.rm.ingv.it/event/7299421
19	2.8	11	41.97	Aug. 28th	4:40 am	http://cnt.rm.ingv.it/event/7318921
20	2.8	9	45.44	Aug. 29th	6:04 am	http://cnt.rm.ingv.it/event/7367651
21	2.8	10	44.94	Aug. 28th	2:44 pm	http://cnt.rm.ingv.it/event/7338361
22	2.7	9	61.41	Aug. 28th	10:00 pm	http://cnt.rm.ingv.it/event/7352691
23	2.7	10	38.31	Aug. 28th	12:44 am	http://cnt.rm.ingv.it/event/7334991
24	2.6	10	34.69	Aug. 27th	6:55 pm	http://cnt.rm.ingv.it/event/7297391
25	2.6	10	39.20	Aug. 28th	5:34 pm	http://cnt.rm.ingv.it/event/7342961

TABLE 2. Comparison metrics between experimental and predicted signal for all the measuring positions and for all the considered windows related to the 4.4 ML earthquake. Bold text is used for the measuring point 2Ay analysed in Fig. 10.

		Reference channel									
Window	Comparison. metric	1Ax	1Ay	1Bx	2Ax	**2Ay**	2Bx	Ta_1	Ta_2	Tb_1	Tb_2
1st 1.46 - 7.08 [s]	M_φ	0.98	0.99	0.98	1.00	**1.00**	0.99	0.99	1.00	1.00	1.00
	M_p	0.93	1.00	0.96	0.89	**0.99**	0.93	0.90	0.81	0.87	0.82
	M_{rms}	0.93	0.94	1.00	0.93	**0.97**	0.99	0.89	0.85	0.94	0.84
	M_w	0.92	0.91	0.91	0.90	**0.93**	0.90	0.86	0.87	0.87	0.87
	M_{wf}	0.86	0.83	0.82	0.81	**0.81**	0.81	0.82	0.83	0.82	0.81
2nd 7.08 - 11.72 [s]	M_φ	1.00	1.00	1.00	1.00	**1.00**	1.00	1.00	1.00	1.00	1.00
	M_p	0.96	0.95	0.96	0.97	**0.92**	0.99	0.95	0.92	0.96	0.93
	M_{rms}	0.98	0.97	0.96	0.99	**0.97**	0.97	0.96	0.92	0.98	0.92
	M_w	0.97	0.97	0.94	0.97	**0.97**	0.93	0.89	0.88	0.91	0.87
	M_{wf}	0.92	0.94	0.88	0.92	**0.94**	0.88	0.82	0.83	0.84	0.88
3rd 11.72 - 14.65 [s]	M_φ	0.99	0.97	0.99	0.98	**0.99**	0.99	0.99	0.98	0.81	0.98
	M_p	0.90	0.95	0.83	0.87	**0.93**	0.95	0.81	0.91	0.82	0.97
	M_{rms}	0.95	0.92	0.85	0.95	**0.91**	0.85	0.99	0.97	0.89	0.95
	M_w	0.95	0.93	0.87	0.94	**0.92**	0.86	0.88	0.88	0.88	0.88
	M_{wf}	0.93	0.92	0.86	0.91	**0.91**	0.87	0.85	0.85	0.87	0.84
4th 14.65 - 27.10 [s]	M_φ	1.00	1.00	1.00	1.00	**1.00**	1.00	0.87	0.99	1.00	0.99
	M_p	0.96	0.99	0.92	0.89	**0.99**	0.88	0.80	0.84	0.81	0.81
	M_{rms}	0.98	0.98	0.92	0.97	**0.98**	0.91	0.80	0.83	0.89	0.83
	M_w	0.97	0.97	0.95	0.96	**0.97**	0.95	0.86	0.86	0.87	0.86
	M_{wf}	0.80	0.83	0.87	0.88	**0.80**	0.87	0.84	0.84	0.85	0.84
5th 27.10 - 40.00 [s]	M_φ	1.00	0.98	1.00	1.00	**0.98**	1.00	0.98	0.81	0.86	0.81
	M_p	0.95	0.96	0.93	0.91	**0.98**	0.94	0.81	0.81	0.88	0.82
	M_{rms}	0.96	0.99	0.97	0.95	**1.00**	0.97	0.82	0.82	0.92	0.81
	M_w	0.97	0.97	0.95	0.96	**0.97**	0.94	0.86	0.86	0.87	0.87
	M_{wf}	0.98	0.98	0.84	0.91	**0.95**	0.81	0.83	0.83	0.92	0.85