Experimental dynamic identification of strategic building structures under real seismic loading

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ABSTRACT: Public buildings such as hospitals, schools, churches and city halls constitute an asset of strategic importance for the life of a community, whose vulnerability to natural hazards can be advantageously evaluated by means of permanent vibration monitoring systems. In the event of an earthquake, these systems can track the actual structural dynamics, allowing the experimental identification and calibration of numerical models for assessment or rehabilitation purposes. This concept has recently inspired the foundation, within the Italian Department of Civil Protection, of a national network for the permanent seismic monitoring of more than one hundred strategic buildings. Four of these buildings have been selected as case studies in the framework of the inter-university Italian DPC-ReLUIS 2010-2013 Project to the aim of dynamic characterization. This paper briefly introduces the said case studies and exemplifies the identification procedure on one of them, the city hall of San Romano in Garfagnana.

1 INTRODUCTION

The numerous recent collapses occurred to existing buildings during medium to severe earthquakes show the potential of structural monitoring for maintenance and management of the building heritage. Whilst the design of new constructions widely relies on structural models which ensure a direct measure of structural reliability, the specific characters of existing constructions require that adequate models be derived and updated on an experimental basis, which can be profitably obtained through the installation and management of a permanent monitoring system. The growing use of these systems allows collecting response data during seismic loading. In the event of an earthquake, these systems can track the actual structural behavior, allowing for the calibration of numerical models, which can be used for assessing the structural seismic safety as well as for designing possible rehabilitation interventions.

This concept has inspired the foundation, starting at the end of the ‘90s and through various subsequent further implementations, within the Italian Department of Civil Protection (DPC), of the Structures Seismic Observatory (OSS), a national-wide network for the permanent monitoring of the seismic response of more than one hundred strategic public buildings in Italy (Spina et al. 2011). This network allows a rapid evaluation of the damage induced by seismic actions on monitored structures and on neighboring similar buildings, helping DPC in planning and managing emergency activities in the earthquake’s aftermaths. The OSS network, includes: (i) the primary sub-network, comprising 105 strategic public buildings (schools, hospitals and city halls), 10 bridges and a few dams, all intensely instrumented with a dynamic permanent monitoring system (16-32 accelerometers each), and (ii) the secondary sub-network, comprising 300 buildings, all equipped with a simplified monitoring system (7 accelerometers each).

Four of the buildings included in the main sub-network, monitored during the Lunigiana/Garfagnana event (January 27th, 2012), have been selected, in February 2012, as case studies in the framework of the inter-university Italian DPC-ReLUIS 2010-2013 Project, and commended to the Department of Structural, Geotechnical and Building Engineering at Politecnico di Torino, as a partner Unit, to the aim of dynamic characterization. This paper briefly introduces these four case studies and exemplifies the modal identification and model-updating procedure on one of them, namely the city hall of San Romano in Garfagnana.

2 THE FOUR SELECTED CASE STUDIES

The four case studies are listed in Table 1 and depicted in Figure 1. They have all been monitored by DPC during the seismic event which stroke the Lunigiana/Garfagnana area on the 27^{th} of January, 2012, whose main properties are summarized in Table 2. For each of the four buildings, a dynamic identification has been performed using the acceleration data recorded during the said seismic event and a finite element (FE) model has been accordingly constructed and updated. For the sake of brevity, the identification and calibration procedure is exemplified in what follows for the first building only, i.e. the city hall of San Romano in Garfagnana.

Table 1. The four selected case-study buildings (buildings description provided in Figure 1)

	Building (a)	Building (b)	Building (c)	Building (d)
Structural typology	Masonry	RC frames	RC shear walls	Masonry
Year of construction	1930	1972	1972	1359 (rebuilt 1761)
Volume (m^3)	2990	1687	7726	1764
Height (m)	14.8	15.8	16.3	9.5 (bell tower 23)
# of accelerometers	20	16	22	26
PGA along X/Y (g)	0.023/0.018	0.024/0.031	0.035/0.020	0.035/0.053

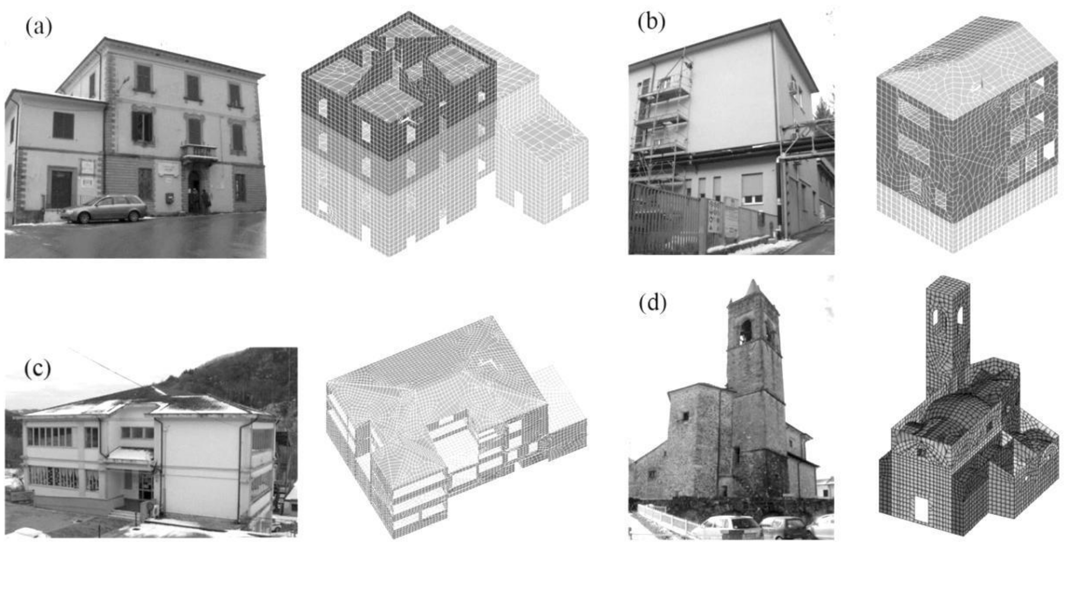


Figure 1. Four selected case-study buildings: (a) City hall of San Romano in Garfagnana ; (b) Hospital of Castelnuovo di Garfagnana; (c) City hall of Casola in Lunigiana; (d) Church of S. Caterina a Collegnago

Table 2. Main parameters of the Lunigiana/Garfagnana event (27 Jan. 2012)

Epicenter	UTC Time	Magnitude
Lat./long.: 44.483°/10.033°	27/01/2012	5.4
Depth: 60 km	14:53:14	

3 THE CITY HALL OF SAN ROMANO IN GARFAGNANA

3.1 The building

San Romano in Garfagnana is located in a medium-to-high seismicity area (seismic zone 2 according to the OPCM 3274 Seismic Code, 2003). The city hall consists of a main building and two adjacent smaller units constructed at a later time (Figure 2a). The main building is a 4-storey masonry structure built on reinforced concrete foundations, having a 19.2x11.4 m^2 rectangular plan and a 14.8 m height. Built in the '30s, it was seismically and statically retrofitted in 1998. Foundations were sandwiched between new reinforced concrete curbs, in order to achieve a more uniform distribution of soil pressures. Main masonry walls were plated with reinforced cement mortar, in order to increase shear strength. The original masonry walls distribution was recovered through reconstructing those elements which had been removed through the years. Floors were stiffened and strengthened, and their connection with masonry improved. In order to enhance structural regularity, a structural joint was realized between units 2 and 3, while units 1 and 2 were connected through steel bars' reinforcements inserted in adjacent walls. A survey on the properties of the constituent materials indicates a characteristic compressive strength of 115 MPa for the stone blocks and of 7.2 MPa for the masonry panels.

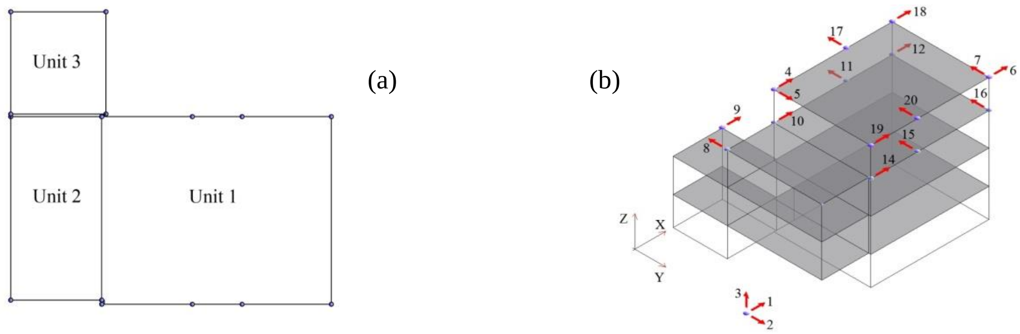


Figure 2. City Hall of San Romano in Garfagnana: (a) planar view; (b) sensors position and orientation

3.2 The monitoring system

OSS permanent monitoring system consists of 20 force-balance uniaxial accelerometers. 17 accelerometers are installed along the building height and the remaining 3 accelerometers, constituting the ground reference channels, are placed inside an underground inspection hole. The position and orientation of the sensors are shown in Figure 2b. Among the 17 accelerometers deployed along the elevation, 16 are installed in units 2 and 3 (attached to the floors at the first and second levels) and only one is deployed in unit 3 (along the X direction). The acquisition system incorporates an AD converter and is connected with a GPS receiver for synchronization with the international UTC time.

4 THE NUMERICAL MODEL

Based on original archive documentation, a preliminary FE model has been constructed in order to highlight possible critical issues in the structural dynamic response and help interpreting processed signals. The model consists of bi-dimensional plate- and shell-type finite elements grouped in macro-elements. The wooden roof has been modeled only in terms of its inertial contribution, as a uniformly distributed mass on the beams which crown the masonry walls.

Masonry properties have been deduced partly from destructive and non-destructive tests and partly from typical values suggested by the literature and by current Italian codes, depending on the outcome of visual inspections. As a result, the masonry elastic modulus has been assumed as 1.5 GPa (Table 4). The structural nodes are clamped at the foundations (about 3 m under the ground level for unit 1 and at the ground level for units 2 and 3) and, in order to simplistically account for soil-structure interaction at the lateral underground walls (unit 1), the thickness of the latter is doubled in the baseline model.

The efficacy of the structural joint located between units 1 and 3 is highly uncertain because of the lack of documental information and because of the presence of only one sensor in unit 3. The joint is therefore modeled as a stripe of shell elements with the same thickness as the masonry walls and a reduced elastic modulus, manually calibrated to 0.10 GPa in order for the analytical modes' sequence to match the experimental one. The subsequent sensitivity analysis (as explained in § 6.3) confirms the small influence of such a parameter to the overall modal error.

5 DATA ACQUISITION AND MODELLING

The analyzed experimental data are represented by the 20 acceleration signals recorded during the Lunigiana/Garfagnana earthquake. Figure 3 reports the time history and the acceleration spectra of the three input components, as recorded by the three ground reference sensors, respectively along the X, Y and Z directions. The seismic energy content appears to be mostly concentrated in the 2-7 Hz frequency band.

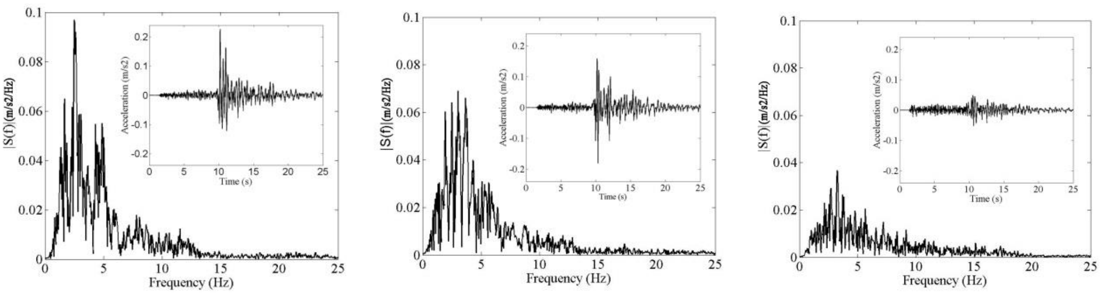
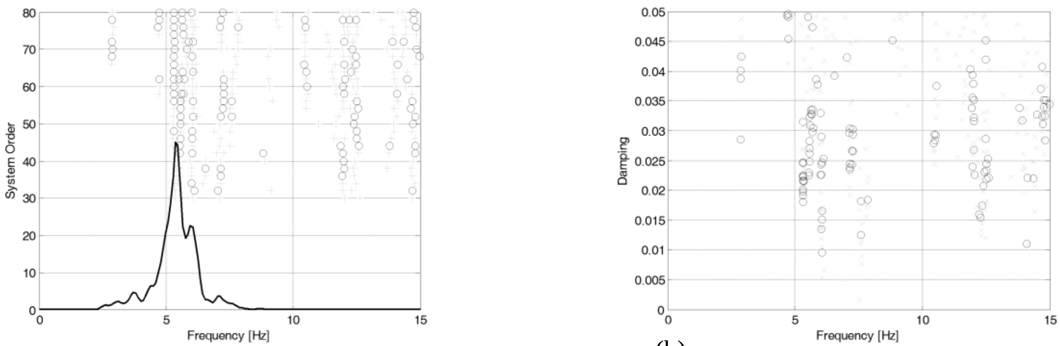


Figure 3. Acceleration spectra (and time-histories) of the three input components

Structural modal identification has been conducted through applying an algorithm belonging to the family of SSI (Stochastic Subspace Identification), implemented in the SDIT3 code (Ceravolo and Abbiati, 2013). The SSI algorithms derive from classic realization theory, extended to stochastic systems. Van Overshee and De Moor (1996) collected and unified in a systematic manner contributions coming from different disciplines: system theory, statistics, optimization and linear algebra. The essence of all SSI algorithms is their capacity of extracting matrices describing a linear system based on subspaces containing projections of data. The technique used for this application is the third algorithm from Van Overshee and De Moor's theorem (1996).

The identification algorithm has been applied to the output signals through progressively increasing the system order. Poles of a given order are classified as stable if all of the following stabilization criteria are satisfied: (i) frequency variation < 2%; (ii) damping variation < 15%; (iii) damping ratio values comprised between 0% and 10%; (iv) MAC (Modal Assurance Criterion) > 95%. Stabilization and clustering diagrams are reported in Figure 4.



(a) (b)

Figure 4. (a) Stabilization diagram ("x": unstable mode; "o": stable mode); (b) Frequency-damping clustering diagram

In the present application, since the structural response is produced by a non-stationary, seismic input, the identified stable poles may include ground motion frequencies. Real structural poles are isolated from ground motion poles through both the comparison with analytical modes of the baseline model (coherence in the mode-shapes and in frequency spacing) and the evaluation of input-output transfer functions. The resulting identified frequencies and damping ratios are reported in the first four columns of Table 3 for the first five modes.

Table 3. Experimental and analytical (updated) modal properties for modes 1 to 5

Mode	Mode type	f_e (Hz)	ζ_e (%)	f_a (Hz)	$(f_a - f_e)/f_e$ (%)	MAC
1	1st flexural Y	5.32	1.8	5.39	1.3%	93%
2	1st flexural X	6.08	1.8	5.76	-5.3%	91%
3	1st torsional	7.17	2.9	7.56	5.4%	82%
4	Local mode	11.98	3.6	11.73	-2.1%	82%
5	2nd torsional	15.65	1.5	15.64	-0.1%	95%

Identified mode-shapes 1 to 4 are represented in Figure 5. It is worth noting that unit 3 seems not significantly involved in the first global modes but provides the largest displacement component in the fourth mode, which is therefore labeled as a local mode. This suggests a very limited participation of this unit to the overall dynamics of the building, and consequently a substantial efficacy of the uncertain structural joint between units 1 and 3.

6 MODEL UPDATING

The parameters of the baseline FE model described above have been updated so that the analytical modal model would match the experimental modal model.

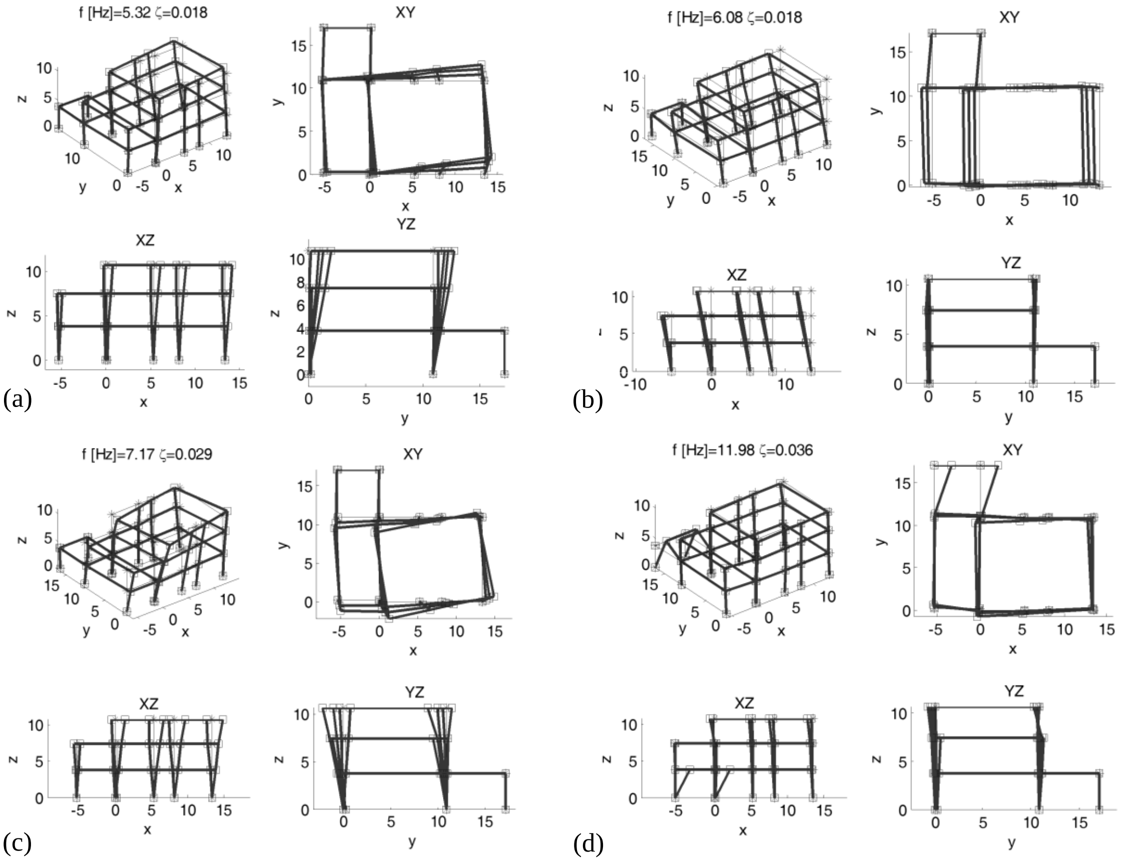


Figure 5. Identified mode-shapes: (a) mode 1 - 1st flexural along Y; (b) mode 2 - 1st flexural along X; (c) mode 3 - 1st torsional; (d) mode 4 - local mode

6.1 *Discretization of the FE model*

Depending on the different age and type of the main parts of the three units, a total of 17 materials (linear elastic and isotropic) are preliminary identified in the baseline model. In order to reduce the number of updating variables, parameters which are likely to be less uncertain and less influential on the global dynamic behavior are excluded from updating. The model is therefore parted in the 6 homogeneous zones represented in Figure 6a.

6.2 *Objective function and sensitivity analysis*

The objective (or error) function selected for updating, and therefore used for the sensitivity analysis as well, is defined as:

$$\varepsilon(\mathbf{x}) = \sum_{i=1}^{m} \left[\alpha_i \left(\frac{f_{ai} - f_{ei}}{f_{ei}} \right)^2 + \beta_i \left(\frac{1 - \sqrt{MAC_i}}{MAC_i} \right) \right] \tag{1}$$

where $\mathbf{x}$ is the updating parameter vector, f_{ai} and f_{ei} are the analytical and experimental i-th natural frequencies, MAC_i is the Modal Assurance Criterion between the i-th identified mode-shape and the corresponding i-th analytical mode-shape, α_i and β_i are the weight factors for the frequency and the mode-shape errors, respectively.

The sensitivity analysis is based on the local sensitivity index defined by Saltelli et al. (2000) as:

$$\hat{S}_i = \frac{\ln(\varepsilon) - \ln(\varepsilon_0)}{\ln(x_i) - \ln(x_{i0})} \tag{2}$$

where x_{i0} and $x_i = x_{i0} \cdot (1+\delta)$ are, respectively, the initial nominal value of the i-th updating parameter and its value incremented of a given percentage δ (here assumed equal to 5%), ε_0 is the error corresponding to the nominal model, and ε is the error corresponding to equaling all parameters to their nominal value while setting the i-th parameter to x_i.

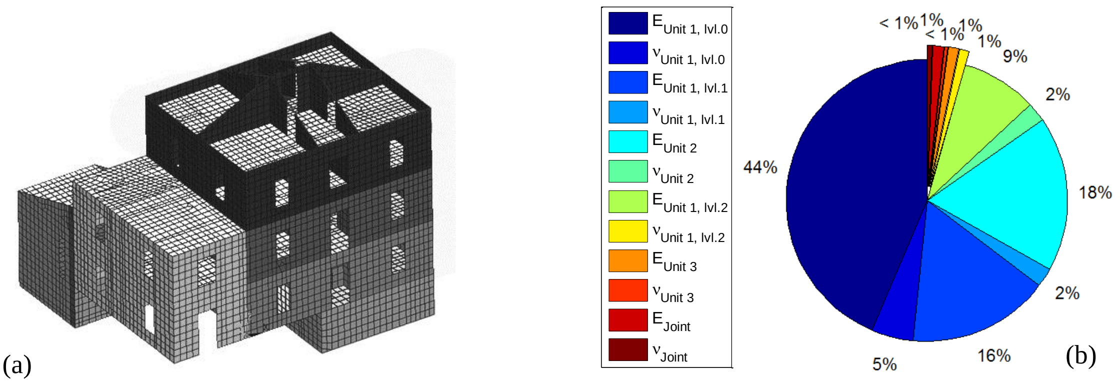


Figure 6. (a) FE model discretization in 6 homogeneous zones; (b) and relative sensitivity index (right)

Expressed in terms of percentage values, the relative sensitivity index is reported in Figure 6b. The elastic modulus of the lower stories appears clearly predominant.
Based on the sensitivity analysis, the four most significant parameters have been chosen as the updating parameters. These parameters are summarized in Table 4, with their initial nominal values reported in the second column.

Table 4. Updating parameters. Second column: nominal values. Third column: updated values

Part	Initial E (GPa)	Updated E (GPa)
1) Masonry of unit 1 – Level 0	1.50	1.84
2) Masonry of unit 1 – Level 1	1.50	1.60
3) Masonry of unit 1 – Level 2	1.50	1.03
4) Masonry of unit 2	1.50	0.78

The model-updating procedure has consisted in the numerical minimization of the objective function ε in Eq.(1) by means of a genetic algorithm, using 15 generations and a population of 300 individuals. Weights α_i and β_i are taken as 0.8 and 0.2, as suggested by Merce et al. (2007). The updated values of the five selected parameters are reported in the third column of Table 4.

The optimal solution is reported in the last three columns of Table 3, where a satisfactory agreement between the analytical and the experimental modal models can be appreciated. MAC values between analytical and identified modes, in particular, are always greater than 80%. The first four mode-shapes of the updated model are reported in Figure 7. Figure 8 shows the good agreement between the experimental and the simulated response to the earthquake input.

7 CONCLUSIONS

The experimental dynamic identification of a strategic building belonging to the OSS permanent seismic monitoring network is here reported as an example of a larger pilot study including a total of four public buildings monitored during a recent earthquake. The pilot study shows that, if supported by a baseline structural finite element model, conventional output-only identification techniques, although ideally conceived for stationary input conditions and linear systems, are a valid tool for the dynamic characterization of structures subjected to small-to-medium ground motions. The study provides useful indications on how to improve the monitoring actions, e.g. sensors' position, and experimentally updated numerical model which can be of help in assessing the current structural seismic safety and in planning future interventions. The procedure will be extended to wider sets of buildings in the near future.

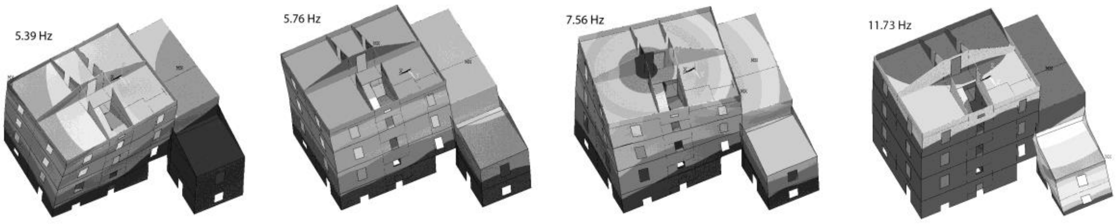


Figure 7. Four first mode-shapes for the updated model

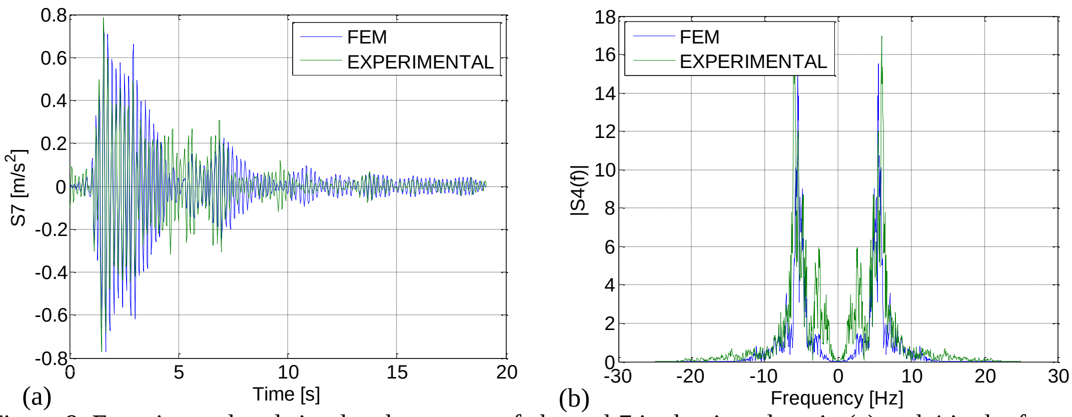


Figure 8. Experimental and simulated response of channel 7 in the time domain (a) and 4 in the frequency domain (b) under the measured ground motion.

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