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Soil-structure interaction effects on the dynamic behaviour of a masonry school damaged by the 2016–2017 Central Italy earthquake sequence

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ABSTRACT: The paper investigates the potential relevance of soil-foundation-structure interaction for the dynamic behavior of a two-story masonry school, which is located in the alluvial valley of Visso (Italy) and was severely damaged by the 2016-2017 Central Italy earthquakes. The main periods and deformed shapes of the structure were identified from the response under ambient noise, recorded on site by the Italian Seismic Observatory of Structures. Modal analyses were performed on 3D models of the fixed-base structure and of the structure placed on springs simulating the soil-foundation impedance. Geometrical and mechanical parameters were accurately calibrated against data acquired from a previous structural survey and a soil investigation carried out during the seismic microzonation of the village. The fixed-base structure model failed to reproduce the dynamic behavior recorded on site, whereas a successful comparison was found between the experimental data and the predicted main vibration periods and modal shapes of the compliant-base structure.

1 INTRODUCTION

Soon after the first main shock event of the 2016-2017 Central Italy seismic sequence, extensive site inspections performed by the Geotechnical Extreme Events Reconnaissance (GEER) group (Stewart et al. 2016, Sextos et al. 2018) revealed that the most damaged towns were located on the top of rock hills or in the valleys. An emblematic case was the municipality of Visso in the Marche region, where the more recently urbanized area is settled on a 40m-thick alluvial layer placed on a stiffer marlstone formation, called Scaglia. Site amplification effects were observed during the 30 October 2016 event (M = 6.5), with the highest degree of damage affecting the buildings founded on loose soil deposits. By contrast, minor damage was detected on structures founded on outcrops of the Scaglia formation.

Among other buildings damaged in Visso, the "Pietro Capuzi" school appeared as a case study of particular interest for possible effects of soil-foundation-structure interaction affecting its dynamic behavior (see Sect. 2). The school was monitored as a strategic building by the Italian Seismic Observatory of Structures - OSS (Dolce et al. 2017). Figure 1a shows the position of permanent accelerometers installed on the structure. Digital records of its dynamic behaviour under ambient noise (data set I) as well as of its response to the whole seismic sequence (data set II) were obtained and processed within the ongoing research activities of ReLUIS project, funded by the Italian Department of Civil Protection (ReLUIS – Task 4.1 Workgroup 2017). While the first set of recordings allowed for the identification of modal parameters (i.e. natural frequencies, damping ratios and mode shapes) of the undamaged

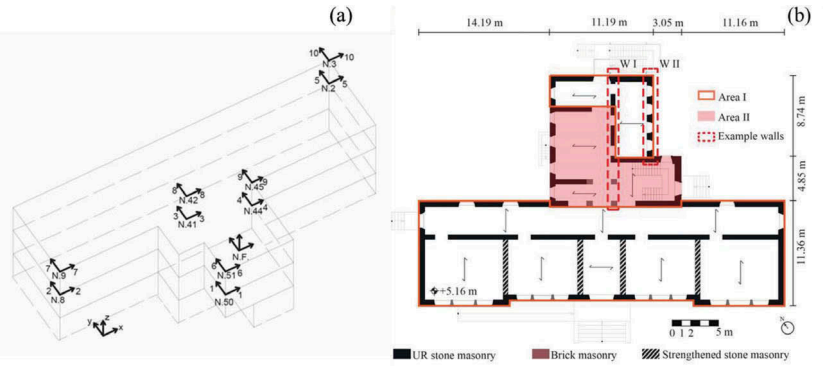

Figure 1. Permanent monitoring system made by OSS (a) and plan view of the "Pietro Capuzi" school (b).

structure, data set II was useful to better understand the effects of cumulative damage. Within this context, the paper illustrates the calibration of a numerical model based on the equivalent frame approach (see Sect. 4.1) which includes the contribution of soil compliance through springs calibrated on basis of an accurate geotechnical characterization (see Sects. 3 and 4.2). As shown in Section 5, such a modelling feature was essential to obtain a good agreement with data set I from the environmental vibration tests.

2 DESCRIPTION OF THE CASE-STUDY AND OBSERVED DAMAGE

The Capuzi school is a masonry structure composed of two stories and an attic covered by a pitched wooden roof; a small extent of the plan presents also an underground level (Area II in Figure 1b). While the vertical layout of the structure is regular, the building has an irregular plan and extends over a 620 m^2 floor surface, resembling a flipped "T" (Figure 1b). The building was built around 1930 but in the '1990s it was subjected to some extensive interventions because the 1997 Umbria-Marche earthquake sequence caused some damages (mainly in the stairwell). Those interventions mainly consisted of mortar injection through some internal walls (indicated as "strengthened stone masonry" in Figure 1b), the stiffening of front openings with steel bracings opened at the bottom, the insertion of some tie-rods, the improvement of the walls-to-roof connection and the replacement of part of the timber roof structure, deteriorated due to aging.

The walls are mainly characterized by a two-leaf stone masonry in which the stones are shaped to fit together in a quite regular way. Diaphragms are composed by a concrete-masonry flooring system consisting of one-way or two-way hollow block slabs, with an exception for the attic that was made by iron beams and thin, hollow clay bricks. Figure 1b indicates the main orientation of floor systems at the second floor level. Reinforced concrete (r.c.) tie beams are present at all stories. The timber roof is a typical "Piemontese" type and the roofing is composed by hollow flat tiles and a thin r.c. slab. The structure is founded on simple enlargements of its loadbearing walls, which were only slightly embedded in the alluvial sediments.

The 2016-2017 Central Italy seismic sequence severely and repeatedly hit the school that turns out to be an emblematic example of structural damage accumulation. The building mainly showed a global response, activating the in-plane behaviour of loadbearing walls with damage concentrated in piers (particularly along the Y direction, as shown in Figures 2a and b) and spandrels. Damage was first produced by the 24 August 2016 shock and was further aggravated by the 26 and 30 October shocks. After the M6.5 26 October mainshock, partial collapse phenomena involved some perimeter walls due to the activation of an overturning mechanism (Figure 2c). Some floors suffered a partial collapse too. An extensive description of structural details of the building and of the observed damage, including its reconstruction after each mainshock, is illustrated in ReLUIS – Task 4.1 Workgroup (2017) and Cattari & Sivori (2019).

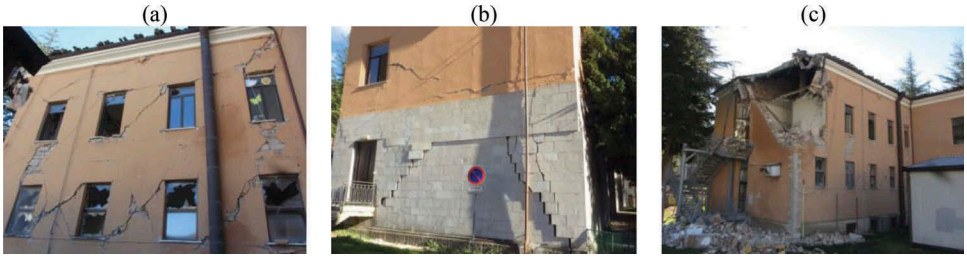

Figure 2. In-plane damage to the NW wall (a) and shear failure of a pier (b); out-of-plane partial collapse of the rear body (c). All photos refer to the damage surveyed on 8th December 2017.

3 GEOTECHNICAL CHARACTERIZATION OF THE SUBSOIL AT THE SCHOOL SITE

The geological setting of the area surrounding Visso village is shown in Figure 3 (Stewart et al. 2016). The outcropping formations belong to the Cretaceous Miocene basinal succession made of Scaglia Rossa (SAA), Scaglia Variegata (VAS), Scaglia Cinerea (SCC) and Bisciaro (BIS), from bottom to top. They are organized in a monoclinal architecture, dipping to West and crossed by normal faults.

The town is set in the depressed area of the Nera river valley, where quaternary alluvial or eluvio-colluvial sediments and widespread slope deposits cover the basinal successions. The thickness of the covering layers varies from few meters to 40 m, reached below the most recently urbanized area. Extensive measurements of ambient noise were performed in the village to identify the soil natural frequency through the HVSR technique. The higher depth of loose coverings in the valley was confirmed by lower resonant frequencies with respect to those recorded in the historical center (Figure 3).

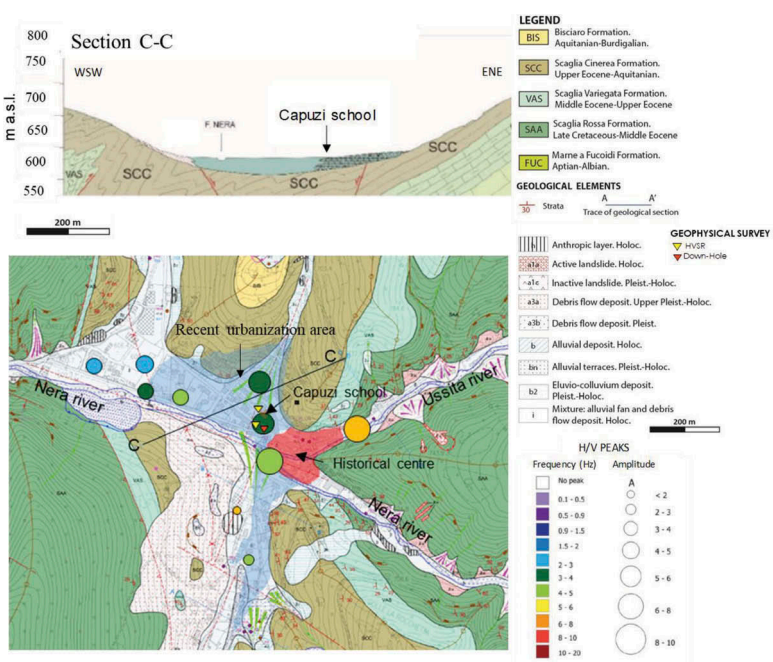

Figure 3. Location of Capuzi school on the geological map and cross section of the Visso village area (modified after Sextos et al. 2018).

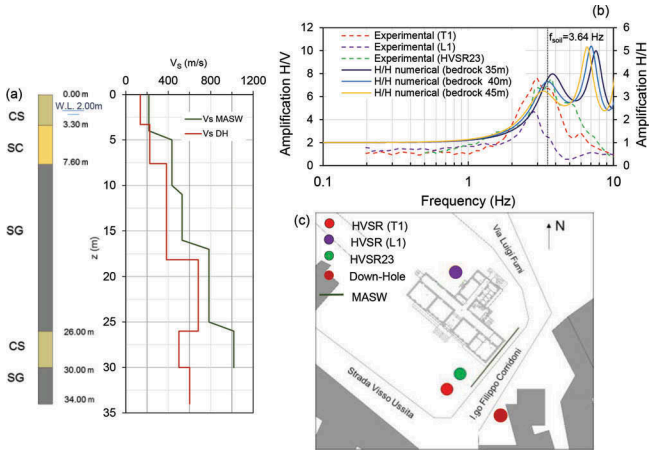

Figure 4. Soil profile and V_S profiles measured through DH and MASW (a), numerical and experimental (HVSR) amplification functions vs. frequency for different bedrock depths (b), plan view of school and location of the surveys (c).

The location of Capuzi school is shown in Figure 3, whereas Figure 4 shows the plan of the first floor, together with field investigations executed during the seismic microzonation study following the recent earthquakes. A borehole was drilled down to 35 m, highlighting the dominant presence of a silty-sandy gravel layer (SG), in which clayey silt (CS) and silty clay (SC) lenses are locally interbedded, as shown in Figure 4a. The water table was intercepted 2 m below the ground level. A down-hole test, DH, was performed in the borehole, leading to the profile of shear wave velocity, V_S, shown in Figure 4a. Overall, V_S tends to increase with depth, showing a significant impedance contrast around 18 m and an inversion around 26 m, exactly where the silt lenses are thicker and closer to each other. The DH results are in fair agreement with the V_S profile measured through a MASW test close to the borehole, again shown in Figure 4a.

Table 1 summarizes physical and mechanical soil properties. The values of shear wave velocity resulting from the DH test were assumed for the layered subsoil model. The soil unit weight, γ, the Poisson's ratio, ν, and the shear wave velocity of the bedrock were inferred from data collected in the seismic microzonation. Low-strain parameters and the bedrock depth were validated by comparing the experimental predominant frequencies with the resonance value computed through 1D seismic response analyses performed along the same vertical (Gaudiosi et al. 2016). The comparison is shown in Figure 4b with reference to three bedrock depths (35 m, 40 m and 45 m). The best agreement between measured and predicted predominant frequencies was found for a bedrock depth of 40 m, which is consistent with the geological model of the area.

Table 1. Physical and mechanical properties of soils.

	z_{min} (m)	z_{max} (m)	γ (kN/m^3)	V_S (m/s)	G (MPa)	ν -
CS_a	0	3.2	20	136	38	0.4
SC_b	3.2	8	20	226	104	0.4
SG_a	8	18	21	383	314	0.3
SG_b	18	26	21	683	999	0.3
CS_b	26	30	20	500	510	0.4
SG_c	30	40	21	602	776	0.3
Bedrock	40	–	22	1300	3790	–

4 NUMERICAL MODELING

4.1 *Structural model*

The structural knowledge acquired from the survey together with some pre-existing results regarding the mechanical properties of the masonry (ReLUIS – Task 4.1 Workgroup 2017) permitted to develop a 3D model of the structure based on the Equivalent Frame (EF) approach.

The choice of such a modelling approach is justified by the regular pattern of wall openings in Capuzi school and by the evidences from the actual response, with cracks mainly developed in piers and spandrels (at least until the second shock of 26th October 2016) as assumed by the EF idealization that concentrates in these elements the nonlinear response. The model has been realized with Tremuri program (Lagomarsino et al. 2013) that is particularly efficient also in performing nonlinear dynamic analyses (Cattari et al. 2018), being the latter an essential requirement for the numerical simulation of the actual response of the structure. In the model, masonry panels were idealized through nonlinear beams with lumped inelasticity, while diaphragms are modelled as 3- or 4-node finite elements of orthotropic membrane in plane stress conditions (see Figure 5). The flexural behaviour of the diaphragms and the out-of-plane response of walls were not considered, assuming a global building response mainly governed by the in-plane behaviour. This assumption is consistent with the main behavior exhibited by the building, at least until the M6.5 26 October event, activating the local mechanism in the rear body (see Figure 2c).

Two structural models were developed: the former having a fixed base and the latter with springs that simulate the soil-foundation impedance. The underground storey located in Area II (Figure 1b) was not explicitly modeled, but it influences the values adopted for soil springs as clarified in Section 4.2. The sensitivity of the dynamic response to this assumption is discussed in Section 5. In this section, the results of a preliminary sensitivity analysis aimed at quantifying the role of other epistemic uncertainties in the capacity modeling are discussed. Hence, analysis results are here related only to the fixed-base model of the structure.

Although quite accurate data were available in terms of structural geometry, structural detailing and diaphragms, model uncertainties still may involve various aspects. Some of them can be treated as random variables, such as the mechanical properties of materials, whereas others lead to alternative capacity models (Y_k). In the following, the second typology is investigated assuming for the first one a preliminary set of reference values based on the knowledge level reached about the structure. More specifically, Table 2 outlines the values adopted for the masonry types detected in Capuzi school. They are compatible with the reference range of variation proposed in the Italian Building Code Commentary (MIT 2009) for "cut stone masonry with good bonding" (characterizing most of the walls) and the "brick masonry with mortar joints" (characterizing pillars and some infilled walls). Regarding the first masonry

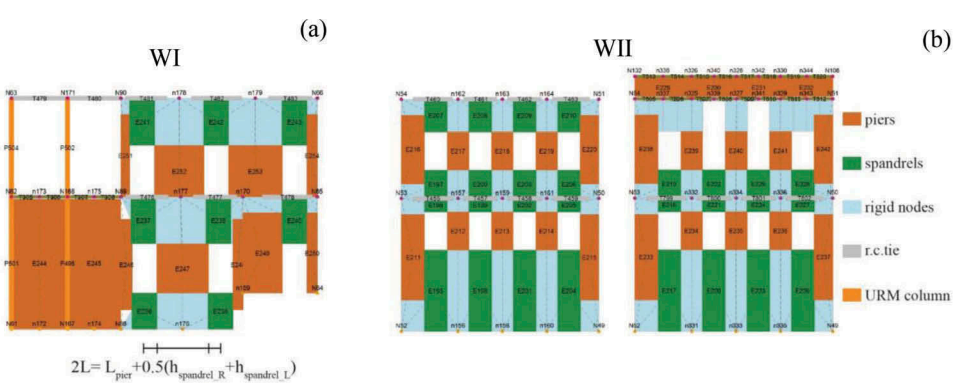

Figure 5. Equivalent frame idealization of some walls: piers with different foundation level (a); different strategies to model the masonry portions in the attic level (as equivalent mass b1 or explicitly b2) (b).

Table 2. Mechanical properties assumed for masonry.

	Cut stone with good bonding (strengthened)	Brick masonry with mortar joints
E [MPa]	1740 (2610)	2250
G [MPa]	580 (870)	750

Table 3. Alternative modelling assumptions considered.

	Alternative modeling options		
	A	B	ΔT_1[%]
Y_1	Equal to opening length	Between that of option A and the distance between consecutive nodes	–3.99
Y_2	Same conventional ground level over all piers	Varying with the actual foundation level	–8.36
Y_3	Attic and roof modeled as equivalent mass	Attic and roof explicitly modeled	+5.53

type, the mean value was assumed, whereas the mean value of the second masonry type was amplified by 1.5 to account for the good quality of mortar. The same factor was adopted to account for the mortar injection made through some vertical walls (Figure 1b). Values assumed for the cut stone masonry are also consistent with some results of double flat jack tests performed on the structure before the seismic event, as available to OSS. Stiffness properties of diaphragms were assumed compatible with those of a r.c. slab with thickness equal to 0.05 m.

Further uncertainties leading to alternative models were considered as follows: the effective length of r.c. tie beams (Y_1); the effective length of piers at the ground level (Y_2); the roof modelling (Y_3). For each one of them, two alternative modeling options were considered, as summarized in Table 3 together with the resulting variation in the period associated with the fundamental mode of the structure. Positive and negative percentages are associated to a shortening or elongation of the period, computed with respect to option A. Uncertainty Y_1 reflects a different possible effectiveness of the actual restraint provided by the masonry on r. c. tie beams at the location of wall openings. Indeed, the two options considered for Y_3 factor do not produce different magnitudes of axial load in piers and inertia forces, even though different equivalent frame idealizations are derived in the top part of walls (see Figure 5b) characterized by the presence or absence of spandrels. The Y_3 uncertainty is relevant for the second step of this research, which will address the numerical simulation of nonlinear seismic response to reproduce the observed damage patterns. A more detailed comparison with the target periods resulting from the post-processing of data set I is illustrated in Section 5, highlighting also that restraints at the base of the structural model play a more relevant role than other model uncertainties.

4.2 *Soil compliance*

As usually adopted in most structural models with compliant base, the foundation and soil of Capuzi school were represented through springs and dashpots associated with each degree of freedom of the foundation. The dynamic stiffness and damping properties were derived from the impedance functions proposed by Gazetas (1991) for an arbitrarily shaped rigid foundation embedded in a visco-elastic half-space, expressed by the general equation:

$$S = K_{stat}k(a_0) + i\omega C(a_0) \tag{1}$$

In the real part of Eq. (1), K_{stat} is the static stiffness and $k(a_0)$ is the dynamic stiffness coefficient depending on the frequency factor $a_0 = \omega B/V_S$, with ω the angular frequency given by $2\pi f$. The imaginary part ($i\omega \cdot C(a_0)$) reflects the damping accounting for both wave radiation

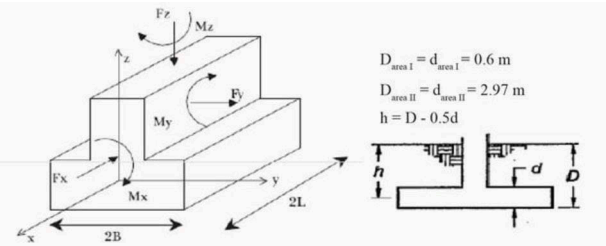

Figure 6. Main parameters required for computation of spring stiffness (adapted from Gazetas 1991).

Table 4. Mean value and standard deviation of the real part of the impedance functions of loadbearing walls.

		K_x (kN/m)	K_y (kN/m)	K_z (kNm)	$K_{r,x}$ (kNm)	$K_{r,y}$ (kNm)
Area I	$\mu_{area\ I}$	2.9E+05	4.1E+05	5.9E+05	3.5E+05	2.8E+05
	σ_{area_I}	0.32	0.45	0.44	1.85	1.38
Area II	$\mu_{area\ II}$	7.9E+05	6.7E+05	1.4E+06	2.3E+06	4.2E+06
	σ_{area_II}	0.21	0.12	0.41	0.08	0.58

and soil hysteresis and viscosity. Since the adopted structural software is not able to implement base-supporting dashpots, at this stage of the work only the stiffness contribution of soil compliance was considered.

In this case, the assumption of a homogeneous half-space for the subsoil is realistic: in fact the depth where the uppermost stiff layer is intercepted (around 18m below the ground level as shown in Figure 4a), is much higher than that affecting the foundation motion, approximately equal to the foundation width, as suggested by Stewart et al. (2003) and Mylonakis et al. (2006).

The geometry of the foundation and the mechanical parameters adopted in the calculation are reported in Figure 6.

The foundation length, 2L, was computed adding the half-length of the spandrel panel to the size of the bearing wall (see Figure 5a), while an enlargement of 0.15 m at each side of the bearing wall was considered to compute the foundation width, B. The value of the embedment, D, was set to 0.6 m for Area I (see Figure 1) and 2.95 m for Area II, where there is the underground level. A full contact was supposed between the foundation and surrounding soil (d/D = 1 in Figure 6), since low-strain levels are mobilized in the identification analysis. Similarly, K_{stat} was computed considering the small-strain shear stiffness of the soil. This latter was derived from the mean shear wave velocity measured in the shallowest 3 m of the first soil layer, corresponding to the depth (approximately 1÷2B) of the soil volume expected to affect the foundation motion (de Silva et al. 2018a). The dynamic stiffness coefficients $k(a_0)$ associated with the main structural period detected on site were then selected. Their values resulted close to unity, due to the low frequency factor associated to such narrow foundations.

Table 4 summarizes the mean value (μ) and the standard deviation (σ) of the impedance functions computed starting from the specific values assigned at the base of each pier of the loadbearing walls; the values computed in the two areas (I and II) of the structure account for the different depth of the embedment.

5 RESULTS OF SSI MODELING

The identified natural periods were compared to those resulting from modal analyses of compliant- and fixed-base models, as shown in Table 5. The numerical results were obtained from two structural models in which options A and B were assumed for the epistemic uncertainty

Table 5. Comparison between the target experimental periods and the simulated ones.

	Mode 1 (s)	err 1 (%)	Mode 2 (s)	err 2 (%)	Mode 3 (s)	err 3 (%)
T_{Target}	0.3150		0.2663		0.2471	
$T_{Spring\_Y3\_opt.A}$	0.2846	–9.62%	0.2651	–0.43%	0.2494	0.97%
$T_{Spring\_Y3\_opt.B}$	0.2677	–15.00%	0.2582	–3.04%	0.2341	–5.25%
$T_{Fixed\_Y3\_opt.A}$	0.2058	–34.65%	0.1935	–23.84%	0.1798	–23.14%
$T_{Fixed\_Y3\_opt.B}$	0.1955	–37.91%	0.1878	–29.46%	0.1715	–30.56%

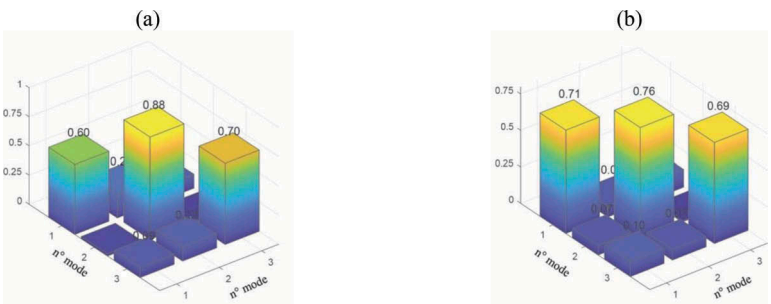

Figure 7. MAC values in case of models with springs and Y_3-option A (a) and spring and Y_3-option B (b).

Y_3, and the most reliable option B was set in both cases for Y_1 and Y_2. Results highlight a potential significant effect of soil-foundation-structure interaction on dynamic response of the building, confirming some evidences of the survey made at scale of the whole historical center and of previous numerical simulations (Ferrero et al. 2018).

In fact, the periods of the fixed-base model are too short in comparison with their target values. Besides, adjusting the values adopted for the other random variables (i.e. those of masonry or diaphragms stiffness) would have led to completely unrealistic periods. Conversely, the periods of the compliant-base model are close to their target values and a slight additional calibration of the mechanical properties of masonry allows for obtaining a very good match. Finally, the experimental and numerical modal shapes were compared through the Modal Assurance Criterion (MAC), which increases from zero to unity as the match improves. Figure 7 shows the MAC values obtained for the compliant-base models for the two options assumed for the Y_3 factor, highlighting a satisfactory comparison and values mostly aligned along the diagonal.

6 CONCLUSIONS

This paper has presented the calibration of a 3D model based on the equivalent frame approach of Capuzi school in Visso, Italy, which is an interesting case study provided by the 2016-2017 Central Italy seismic event. The calibration benefited from data of ambient vibration measurements carried out by OSS before the first 2016 main shock and of an accurate geotechnical characterization of the subsoil at the school site. The model provides the basis for the simulation of the actual response through nonlinear dynamic analyses that are a primary module of this research activity. In addition to some modeling improvements, further analyses will be performed, in which the dynamic interaction between the different foundation systems and the foundation flexibility will be taken into account by reducing the foundation stiffness through the curves calibrated by de Silva et al. (2018b) on numerical simulations by Pitilakis et al. (2015). Nonlinear dynamic analyses performed on both fixed and compliant-

base models, as well as on complete models including the subsoil as a layered continuum, will be addressed to confirm the actual SSI role on the complex response of this structure.

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REFERENCES

Cattari, S., Camilletti, D., Lagomarsino, L., Bracchi, S., Rota, M. & Penna, A. 2018. Masonry Italian Code-Conforming Buildings. Part 2: Nonlinear Modelling and Time-History Analysis, *Journal of Earthquake Engineering*, DOI: 10.1080/13632469.2018.1541030.

Cattari, S. & Sivori, D. 2019. Lessons from the 2016 Central Italy earthquakes: the seismic behaviour of two masonry schools in Visso and Caldarola (Marche Region), *Bull Earthquake Eng*, to be submitted.

de Silva, F., Ceroni, F., Sica, S. & Silvestri, F. 2018a. Non-linear dynamic analysis of the Carmine bell tower accounting for the soil-foundation-structure interaction, *Bull Earthquake Eng*, 6(7): 2775–2808.

de Silva, F., Pitilakis, D., Ceroni, F., Sica, S. & Silvestri, F. 2018b. Experimental and numerical dynamic identification of Carmine bell tower in Naples (Italy), *Soil Dynamic and Earthquake Engineering*, 109: 235–250.

Dolce, M., Nicoletti, M., De Sortis, A., Marchesini, S., Spina, D. & Talanas, F. 2017. Osservatorio sismico delle strutture: the Italian structural seismic monitoring network. *Bull Earthquake Eng*, 15(2): 621–641.

Ferrero, C., Lourenco, P.B. & Calderini, C. 2018. 2016–2017 Central Italy Earthquake: Seismic Assessment of "Pietro Capuzi" School in Visso (Marche), *Proc. of the 9th ICCM conference, 6th-10th August 2018, Rome, Italy*, 418–434, ISSN 2374–3948.

Gaudiosi, I., Vignaroli, G., Sirianni, P., Giallini, S., Mori, F., Polpetta, F., Razzano, R., Simionato, M. & Moscatelli, M. 2016. Local seismic response studies in the north-western portion of the August 24th, 2016 MW 6.0 earthquake affected area. The case of Visso village (Central Appennines). *Annals of Geophysics*, 59, Fast Track,5, 2016.

Gazetas, G. 1991. Formulas and charts for impedances of surface and embedded foundations. *J Geotech Eng*, 117: 1363–1381.

Lagomarsino, S., Penna, A., Galasco, A., Cattari, S. 2013. TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings, *Engineering Structures*, 56: 1787–1799.

MIT 2009. Ministry of Infrastructures and Transportation, Circ. C.S.LL.PP. No. 617 2/2/2009, Istruzioni per l'applicazione delle nuove norme tecniche per le costruzioni di cui al Decreto Ministeriale 14 Gennaio 2008, G.U. S.O. n.27 of 26/2/2009, No. 47 (in Italian).

Mylonakis G., Nikolaou S., Gazetas G., 2006. Footing under seismic loading: Analysis and design issue with emphasis on bridge foundation. *Soil Dynamics and Earthquake Engineering*, 26: 824–853.

Sextos, A. De Risi, R., Pagliaroli, A., Foti, S., Passeri, F., Ausilio, E., Cairo, R., Capatti, M.C., Chiabrando, F., Chiaradonna, A., Dashti, S., de Silva, F., Dezi, F., Durante, M.G., Giallini, S., Lanzo, G., Sica, S., Simonelli, A.L. & Zimmaro, P. 2018. Local site effects and incremental damage of buildings during the 2016 Central Italy earthquake sequence, *Earthquake Spectra*, 34(4): 1639–1669.

Stewart, J.P. et al. 2016. Engineering reconnaissance following the 2016 M6.0 Central Earthquake, Ver 2. doi:10.18118/G61S3Z, http://www.geerassociation.org/.

Stewart J P, Seunghyun K, Bielak J, et al., 2003. Revisions to Soil-Structure Interaction Procedures in NEHRP Design Provisions. *Earthquake Spectra*, 19(3):677–696.

Pitilakis, D & Karatzetzou, A. 2015. Dynamic stiffness of monumental flexible masonry foundations. *Bullettin Earthq Eng*, 13: 67–82.

ReLuis – Task 4.1 Workgroup edited by: Cattari, S., Degli Abbati, S., Ottonelli, D., Sivori, D., Spacone, E., Camata, G., Marano, C., Da Porto, F., Lorenzoni, F., Penna, A., Graziotti, F., Ceravolo, R., Matta, E., Miraglia, G., Spina, D.& Fiorini, N. 2017. Report di sintesi sulle attività svolte sugli edifici in muratura monitorati dall'Osservatorio Sismico delle Strutture, Linea Strutture in Muratura, *ReLUIS report, Rete dei Laboratori Universitari di Ingegneria Sismica*, (In Italian)