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NUMERICAL MODELING OF CAVITY-WALL URM BUILDINGS

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ABSTRACT

This paper presents the results of a numerical activity conducted in the framework of a wider research project that aims at assessing the seismic vulnerability of a particular typology of residential buildings in the Groningen region of the Netherlands, the so-called terraced houses. Simplified nonlinear numerical models have been created to replicate the results obtained by a series of unidirectional dynamic shaking-table tests conducted on a full-scale two-story URM building with cavity walls. The specimen was subjected to incremental dynamic tests up to the near collapse conditions with the objective of ascertaining the ultimate capacity and failure modes of the structure. The hysteretic response of the specimen, as well as the cracking pattern induced by the failure mechanisms activated during the tests, were simulated with the aid of an equivalent frame modelling approach making use of macroelements. A MDOF model of the tested building was calibrated to reproduce the outcomes of the laboratory tests, allowing the study of its sensitivity to salient parameters. In this context, some of the capabilities, as well as the limitations of the employed modelling approach in reproducing different aspects of the experimental response are highlighted and further investigated.

KEYWORDS: *cavity wall, macroelement, shaking-table test, terraced house, unreinforced masonry (URM) building*

INTRODUCTION

In the last decades, people living in the Groningen area have been exposed to an increasing number of earthquakes induced by gas extraction and reservoir depletion. Some building damage has been reported after the recent seismic events and there is a general concern that seismicity in the area

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has increased over the last years. Within the framework of a project devoted to the evaluation of the seismic vulnerability of the existing building stock in the area, extensive experimental and numerical research has been devoted to the study of the seismic response of terraced buildings. The terraced houses embody typical modern Dutch residential building construction and constitute one of the most diffused building typologies in the urban environment of the Groningen province. As most of the building stock in the area (almost 90%), they consist of unreinforced masonry (URM), which in the absence of special design features is typically characterized by a poor seismic behavior.

Assessing the fragility of masonry buildings requires reliable modelling of their seismic response. A dedicated project comprising a series of research activities has been running during the recent years at the EUCENTRE laboratory in Pavia, Italy. Special attention has been given to experimental laboratory testing and numerical modelling of URM building typologies, with the aim of supporting the derivation of fragility curves for the masonry structures in question. The testing campaign included both *in situ* mechanical characterization tests, as well as laboratory tests, such as: (i) characterization tests on small masonry assemblies; (ii) in-plane cyclic shear-compression tests [1] and dynamic out-of-plane tests [2] on full-scale masonry piers; and (iii) full-scale unidirectional shaking-table tests on a building prototype [3]. In particular, incremental shaking-table tests were conducted on the end-unit of a two-story terraced house, built with URM cavity walls.

Based on the outcomes of the tests, the seismic response exhibited by the specimen was simulated by means of an equivalent frame modelling approach involving nonlinear macroelements [4][5]. The basic assumptions embedded in the nonlinear models, as well as the comparison between the numerical analyses and the experimental results are demonstrated. This paper presents several topics concerning numerical modelling of URM cavity walls using macroelements. In particular, the work focuses on the effect of discretization of the equivalent frame, the appropriate definition of model mechanical parameters and the effect of different modelling options for flexible diaphragms on the global seismic response of the structure.

SUMMARY OF SHAKING-TABLE TESTS

Overview of the Building Specimen

The building specimen (Figure 1a) was meant to be representative of the end-unit of a Dutch terraced house system of the late 70s, built with URM cavity walls and without specific seismic detailing, according to the local construction practice of the time. This building typology is characterized by wide openings along the front and back longitudinal walls, while solid walls separate the units in the transverse direction. As a result, longitudinal walls are more vulnerable to in-plane seismic excitation than transverse ones. For this reason, the unidirectional shaking-table test was performed in the longitudinal (North-South) direction (see Figure 1b).

The house was designed as a single-room, two-story building with a pitched timber roof and reinforced concrete (RC) slabs that provided rigid diaphragms at the first and second floors. The walls consisted of two URM leaves: the inner loadbearing leaf was made of calcium silicate (CS) brick masonry, while the external leaf was made of clay brick masonry without any loadbearing function. The resulting gap was 80 mm wide and the connection between the two leaves was realized by means of steel ties. The characteristics of the tested building, as well as the results obtained by the dynamic shaking-table tests are extensively discussed in [3].

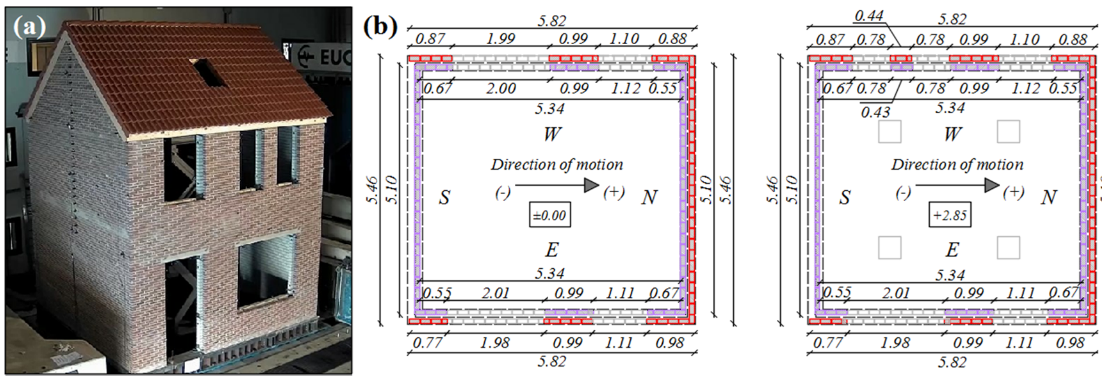

Figure 1: Cavity-wall terraced house specimen: (a) North-West view of the full-scale test building; and (b) ground- and first-floor plan views. Units of m.

Masonry characterization and in-plane cyclic shear tests, performed to complement the experimental campaign, provided useful information regarding the characteristics of the cavity-wall masonry used to build the specimen [1][3]. Data obtained by tests on small masonry assemblies, such as compression tests, bond wrench tests and shear tests on triplets constituted a useful base for the later calibration of the numerical models.

Results of the Shaking-Table Tests

Incremental dynamic tests were performed by applying a series of shaking-table motions of increasing intensity up to the near collapse limit state of the structure. Two input records, EQ1 and EQ2 with PGA 0.10 g and 0.17 g, respectively, were selected as representative of the dynamic characteristics of induced seismicity ground motions. The building tolerated PGA 0.17 g with little damage (global drift θ = 0.07%) and was in a near collapse state after a shaking of PGA 0.31 g (global drift θ = 0.73%). The full in-plane capacity of the longitudinal walls was attained with prevailing flexural/rocking behavior of the masonry piers. An ultimate inter-story drift ratio of θ_1 =0.88% and θ_2 = 0.86% was achieved for the 1st and 2nd story, respectively, while the peak in-plane shear deformation of the roof diaphragm was γ_R = 1.44%. The maximum base shear attained was approximately V_{max} = 139.5 kN (corresponding to base shear coefficient BSC_{max} = 0.25). During the last test (EQ2-200%, PGA 0.31 g), the two longitudinal masonry leaves oscillated independently resulting in the reduced contribution of the external leaf (veneer) to the lateral load-resisting system, and consequently, elongation of the natural period of vibration. The tests were stopped before collapse of the structure, in order to prevent damage to the testing facilities.

MODELING STRATEGY

Within the scope of this study was to provide reliable multiple degree of freedom (MDOF) numerical models for the seismic analysis of cavity-wall buildings. Two-dimensional numerical models of the tested specimen were built and validated with the experimental results. For this purpose, the software TREMURI was employed, where an efficient equivalent frame formulation with nonlinear macroelements has been implemented. The program is capable of running dynamic global analyses of entire masonry buildings neglecting the contribution of the out-of-plane response of walls [4].

The Macroelement Model

The macroelement model, starting from a previously developed model, has been refined in the representation of flexural-rocking and shear damage models, and has proved capable of fairly simulating the experimental response of masonry panels under cyclic loading [5]. The flexural-rocking failure mode of the panel is represented adopting a unilateral contact model, where zero-length springs located at the interfaces follow a bilinear constitutive model in compression with no capacity in tension, allowing the explicit evaluation of how cracking affects the rocking motion (see Figure 2a). The new model includes a nonlinear model for rocking damage that accounts for the effect of limited compressive strength (i.e., toe crushing). It is characterized by an unloading branch with slope equal to the initial stiffness, thus allowing an increased energy dissipation and the ability of modeling deformation accumulation [6]. The shear response of the panel has been modelled through a constitutive law expressing a nonlinear relationship between shear stress and the relative horizontal deformation, as shown on Figure 2b. The macroelement model parameters should be considered as representative of an average behavior of the masonry panel.

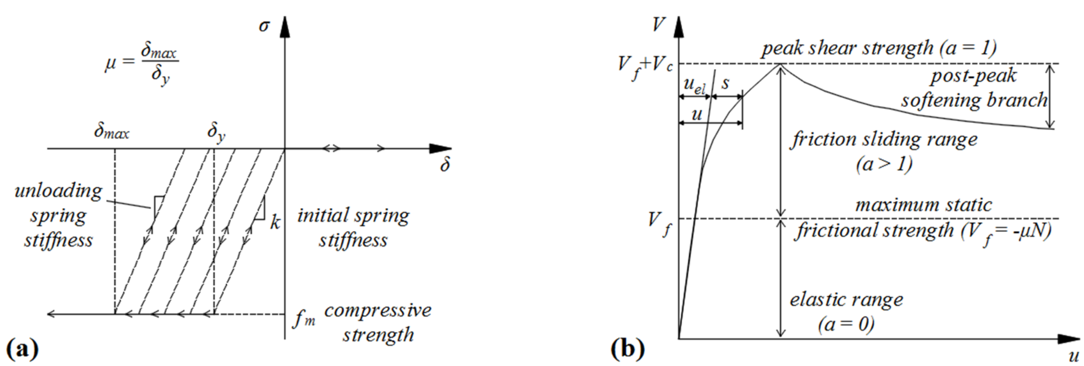

Figure 2: Macroelement constitutive models: (a) interface springs; and (b) shear panel.

NUMERICAL SIMULATION OF SHAKING-TABLE TESTS

The dynamic response exhibited by the tested building during the shaking-table tests has been numerically simulated. Nonlinear time history analyses were performed on calibrated models to reproduce the hysteretic behavior of the specimen, as well as the evolution of the exhibited damage pattern. The numerical models were subjected to acceleration time histories recorded on the shaking-table. Sensitivity analyses were also performed to investigate the influence of several factors on the modelling of the global response, such as: the discretization and geometry of the equivalent frame model; the modelling of the various element connections of the structure; the variability of the mechanical properties obtained experimentally from masonry characterization tests; and the selection of an appropriate value for viscous damping.

Identification of the Equivalent Frame

In the present study, two different geometries of the numerical model were considered in defining the deformable members; the effective height of the piers varies considering the observed crack pattern. In the frame-type representation adopted for the first model, namely "stiff model", the masonry piers were assumed to have the same height with the adjacent openings. This model, having been rather stiff, was used for simulating the shaking-table tests performed up to shaking level of PGA 0.24 g (EQ2-150%). On the other hand, the "flexible model" was shaped accounting for the exhibited damage pattern during the last two tests (EQ2-150% and EQ2-200%), and allowed an improved capture of the damage mechanisms observed. The geometry of the two different used models is illustrated on Figure 3.

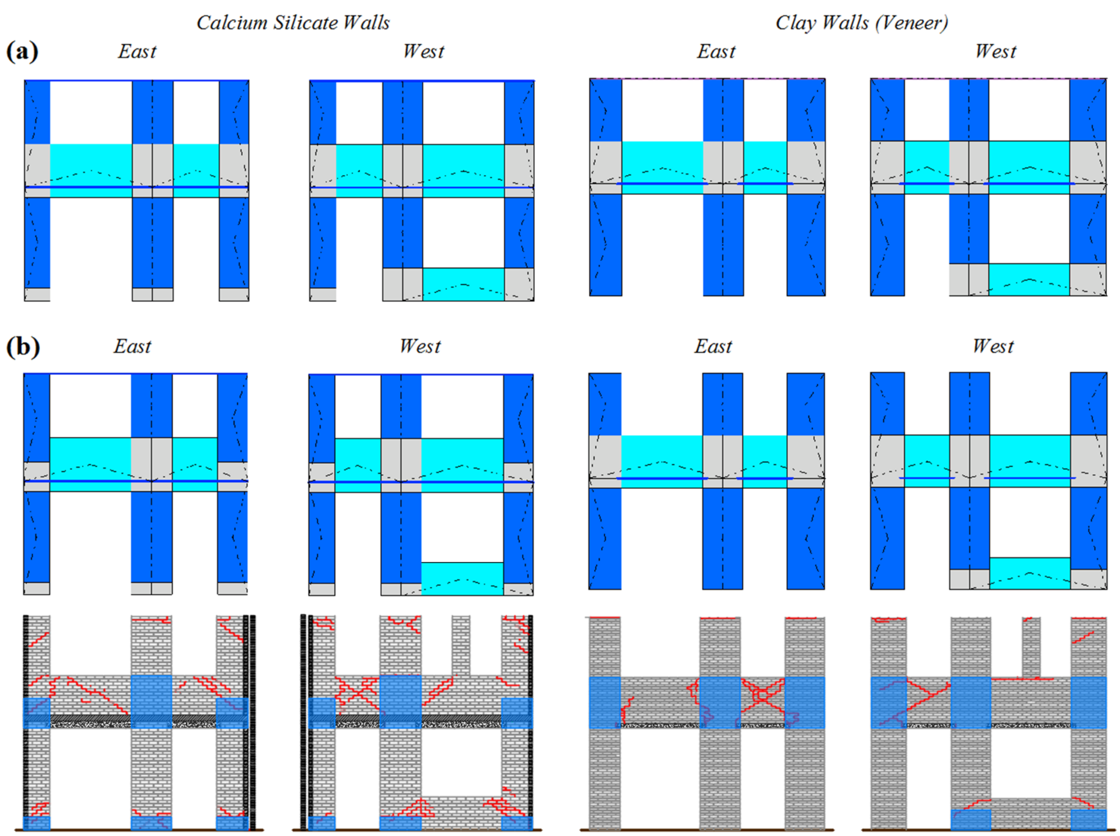

Figure 3: Illustration of the two different geometries adopted for the equivalent frame: (a) "stiff" model; and (b) "flexible" model.

Masonry Mechanical Parameter

The masonry mechanical parameters of the macroelements were selected in consistence with the results of the characterization tests performed on masonry wallettes constructed with materials of the same batches with those used for the construction of the building. The equivalent cohesion,

c_{eff}, and shear friction, μ_{eff}, for each one of the macroelements were computed after accounting for their boundary conditions and their expected stress state in compression, similarly to what has been reported in Penna *et al.* [5]. Refined values of the shear parameters and the modulus of elasticity, E_m, resulted after model calibration against the response of masonry piers subjected to in-plane cyclic test that complemented the experimental campaign [1]. The assumptions on material properties and parameters assigned to the macroelements employed for the models are summarized in Table 1.

Table 1: Masonry mechanical properties for the macroelement models of the test building.

Material property	Symbol	U.M.	Calcium silicate		Clay	
			Piers	Spandrels	Piers	Spandrels
Young's modulus in compression	E_m	MPa	2132	1492	3926	2748
Shear modulus	G	MPa	710	500	1310	916
Density of masonry	ρ	kg/m^3	1835	1835	1900	1900
Compressive strength	f_m	MPa	5.49	5.49	12.72	12.72
Equivalent cohesion	c_{eff}	MPa	0.05	0.02	0.11	0.03
Equivalent friction coefficient	μ_{eff}	-	0.50	0.34	0.60	0.32
Nonlinear shear deformation parameter	Gc_t	-	4	2	4	2
Softening parameter	β	-	0	0	0	0

Diaphragms

The diaphragms were modelled as two-dimensional orthotropic membrane-based finite elements, with four nodes and two displacement degrees of freedom at each node (u_x, u_y) in the global coordinate system. The mechanical properties determining the response of the diaphragms are described by means of four coefficients: the moduli of elasticity, E_1 and E_2, which represent the normal stiffness of the membrane along the two perpendicular directions; the shear modulus, G, which influences the diaphragm's tangential stiffness; the Poisson's ratio, v, and an equivalent thickness, t_{eq}. Table 2 reports the assumptions on the mechanical parameters of the membrane elements implemented for modelling the rigid floors and the flexible roof diaphragm.

Table 2: Mechanical properties of the diaphragms implemented in the numerical models.

Element	Thickness [mm]	Elastic modulus [MPa]		Shear modulus [MPa]	Poisson's ratio [-]
	t_{eq}	E_1	E_2	G	v
Reinforced concrete slabs	160	30000	30000	12500	0.20
Timber roof sheathing	18	11500	11500	25	0.32

The evaluation of the above quantities was rather simple in the case of the RC slabs of the 1st and the 2nd floors, which have been modelled as rigid diaphragms. On the contrary, in the case of the flexible timber roof, the definition of equivalent stiffness properties was not so straightforward. The membrane elements implemented in the TREMURI program exhibit purely elastic behavior. Given the high nonlinearity of the roof response at early stages [3], due to the behavior of nailed connections, the in-plane stiffness of the inclined membrane elements representing the pitched timber roof was modified while running a sequence of incremental dynamic analysis. The initial elastic in-plane stiffness, $G_{initial}$, of the membranes was evaluated based on numerical calculations using formulas proposed in the literature, as well as results obtained by previous experimental investigations concerning the seismic performance of similar timber diaphragms [7-9]. In the sequence of time history analyses that followed, the shear modulus of the membrane elements, G_i, was adjusted accordingly, guided by the trend of stiffness degradation that the roof exhibited in the lab tests, as illustrated in Figure 4a.

Damping

A damping model should be correctly selected to reproduce the inherent damping, as well as the energy dissipation that cannot be captured by the hysteretic model. A classical Rayleigh viscous damping model has been implemented in the TREMURI program, where the damping matrix is computed with the initial stiffness. In the present study, a 5% viscous damping ratio was adopted for simulation of the tests performed at a lower shaking intensity, whereas a value of 2.5% was selected at the last stage of testing (EQ2-200%) during which the specimen exhibited highly dissipative behavior, as adopted also in [10].

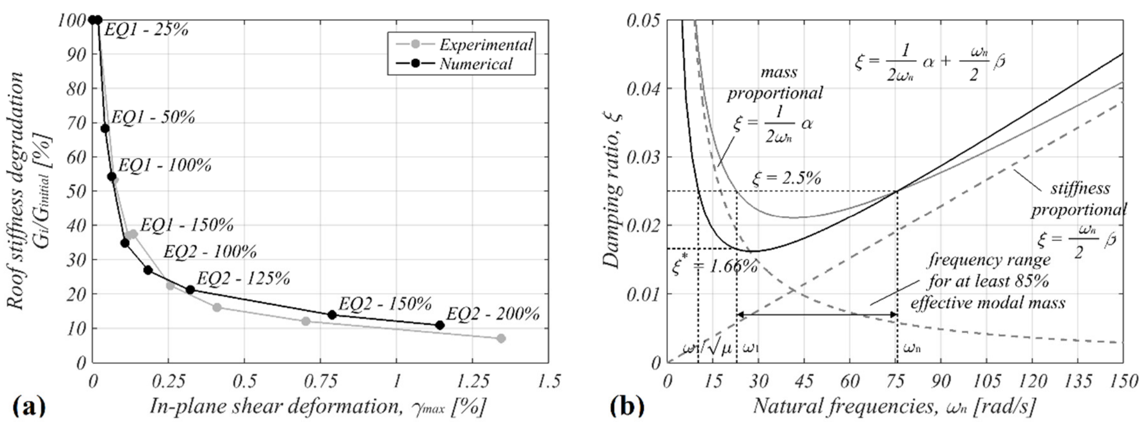

Figure 4: (a) Roof stiffness degradation: comparison of the trend adopted in the numerical model (black) with the experimentally observed one (grey). (b)Variation of model damping ratios with natural frequency in Rayleigh damping (simulation of test EQ2-200%).

A way to treat the problem of unrealistically large damping forces coming from the stiffness-proportional part of the Rayleigh damping model is to specify an artificially low damping coefficient, $\xi^* < \xi$, in the fundamental mode, in light of an expected displacement ductility, μ, as proposed in [11]. This is readily explicable by considering that the more the structure enters the inelastic range, the greater the error induced by the initial overestimation of elastic damping is. The above can alternatively be formulated as specifying the desired damping ratio, ξ, at the lower frequency bound, $\omega_1^* = \omega_1/\mu^{0.5}$, as shown in Figure 4b (test EQ2-200%), supposing that the fundamental frequency of the building is expected to shorten due to nonlinear softening. For this purpose, a value of $\mu = 5$ was used, which approximates the displacement ductility factor achieved in the lab tests.

Additional Modelling assumptions

Particular attention was devoted to the modelling assumptions regarding the connections between the various structural elements to effectively simulate the seismic response of the tested building. The contribution of the cavity ties to the coordinated response of the out-of-plane veneer and loadbearing walls was one of the most important aspects worth exploring. Connecting ties were modelled as nonlinear beam-elements with defined axial strength and stiffness only in tension and compression.

Another feature that determined in a great extent the global response of the model during the numerous performed sensitivity analyses regards the connection provided by means of a thin layer of mortar between the timber wall plate beams and the top of the veneers at the 2nd floor level, as illustrated in Figure 5a. Due to the rigid attachment of these timber beams to the slab (with the aid of threaded bars), this connection proved to provide a perfect coupling of the in-plane response of inner and outer leaves, up to a significant level of seismic excitation. Loss of contact and sliding along the interfaces was first observed when the house was subjected to EQ2-150% (PGA 0.24 g). Since the gravity load transferred from the slab and the timber beams to the top of the veneers was negligible, the residual strength (due to friction) of these interfaces was infinitesimal. The importance to come up with an accurate modelling approach for such highly detailed connections was easily appreciated, and a series of extensive sensitivity analyses was performed.

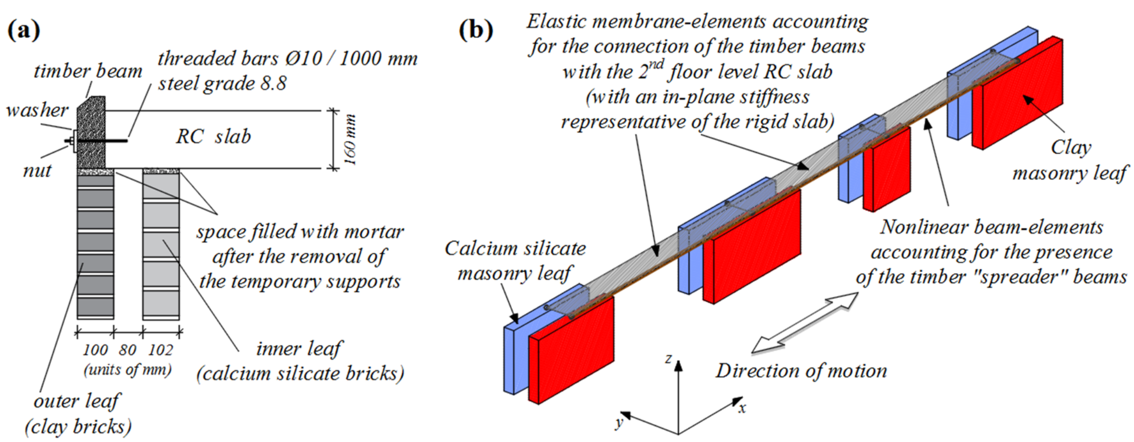

Figure 5: Illustration of the detailed modelling of the connection between inner and outer leaves at the 2nd floor level.

The best matching of the experimental and the numerical response of the model, after the failure of the thin mortar layer, consists in removing the connection between the inner and outer longitudinal leaves provided by the rigid membrane elements shown in Figure 5b. In other words, at this point (test EQ2-150%, PGA 0.24 g) the in-plane stiffness of these membranes was set equal to zero. Analyzing the shear forces developed in the 2nd story piers of the veneer, when the building was subjected to 125% and 150% of EQ2, it was found that an average demand, in terms of shear stress, was equal to 0.093 MPa and 0.145 MPa, respectively. Values within this range could represent the cohesion developed between mortar and timber. A proper response to modelling such delicate connections would be an *ad hoc* implementation of interface elements that, ruled by a cohesion-dependent model (providing resistance until their capacity is reached), could be efficiently incorporated in future developments of the TREMURI program.

Results from the Calibrated Model

The results of the nonlinear dynamic analyses showed good consistency with the experimental response of the tested building. The hysteretic behavior of the structure, together with the progressive stiffness degradation, was numerically reproduced with a fairly good approximation. Comparison of the numerical simulation against the experimental hysteresis curve for testing under EQ2-200% is illustrated on Figure 6a. Figure 6b compares the envelope of the numerical hysteresis loops to the envelope curve obtained by the experimental responses (in terms of maximum attained base shear, $V_{\max}$, versus maximum achieved global drift ratio, θ). The damage pattern predicted by the model at the end of the sequence is illustrated in Figure 7.

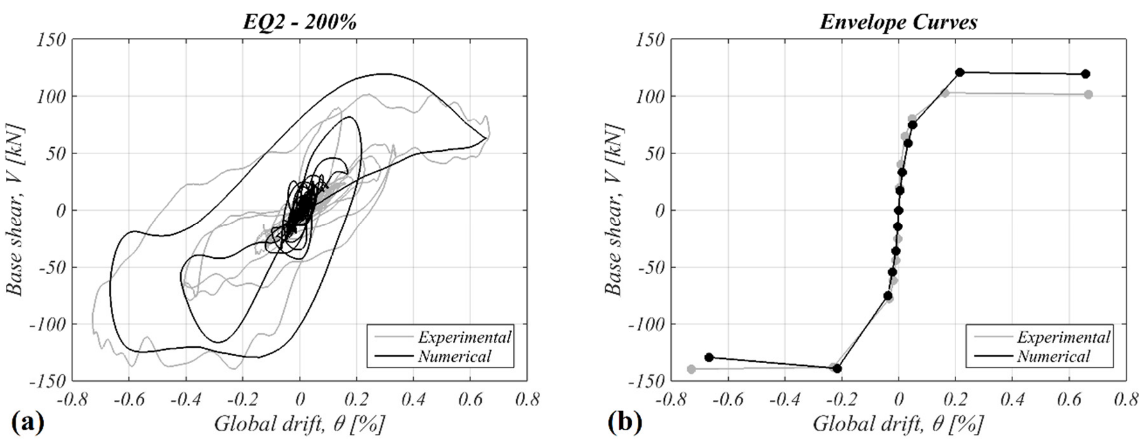

Figure 6: Comparison of the numerical simulation results (black) against the experimental data (grey): (a) global hysteretic behavior during the test EQ2-200%; and (b) envelope curve in terms of maximum base shear versus maximum global drift ratio.

The model provides an accurate simulation of the structural response when subjected to EQ1, capturing in a satisfying degree the global stiffness of the specimen and predicting maximum displacements and base shears similar to those achieved during the shaking-table tests. The model presents the same great efficiency in approximating the response under EQ2, especially when subject to base accelerations up to a level of 0.24 g PGA (EQ2-150%). The further well-aimed response of the model was partially guided by the knowledge of the manifested crack pattern and the local failure of specific connections. In particular, as aforementioned, during the last stage of the incremental dynamic analysis (EQ2-200%), a more flexible model was employed. This model was mainly characterized by the elimination of the connection between inner and outer longitudinal leaves at the 2nd floor level, in this way accounting for the sliding of the timber beams on the top of the veneers. A larger effective height of the corner piers was also considered due to the development of diagonal cracks at their ends. The effect of the uncoupled response of the two longitudinal wall systems was evident in the strongly nonlinear behavior that the structural model exhibited, clearly seen in Figure 6b. The contribution of the longitudinal outer leaves to the global stiffness of the structure was limited to the role of the weak connection ensured by means of the steel ties, and the veneers were sustaining mainly the inertia forces resulting from their self-weight. It should be also stressed that the damping coefficient employed in the simulation of the last shaking-table test was equal to 2.5% (half of the value assumed for all the previous runs), in an attempt to better capture the strongly dissipative hysteretic behavior that the prototype building exhibited, due to the activation of flexural and shear damage mechanisms in the piers and spandrels, respectively.

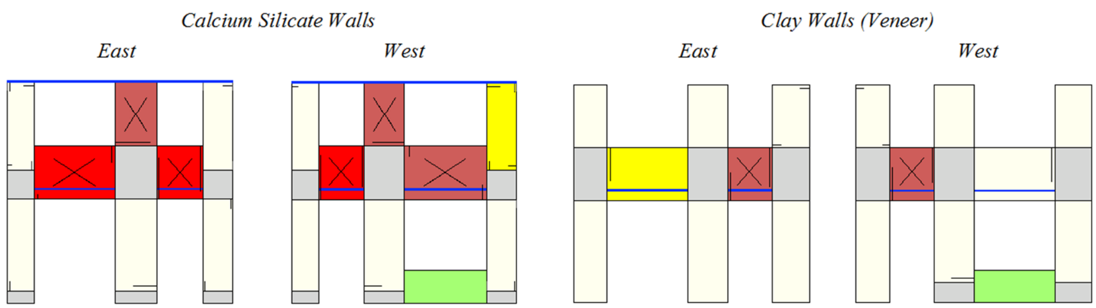

Figure 7: Damage pattern predicted by the numerical model at the end of the shaking-table test sequence.

CONCLUSIONS

Several issues concerning the numerical modelling of the seismic response of terraced houses have been addressed in the framework of using an equivalent frame modelling strategy based on the nonlinear macroelement model, implemented in the TREMURI program. The outcome of the simulations of full-scale shaking-table tests showed that the results obtained from a suitably calibrated numerical model approximate competently the experimental response of the building.

An important consideration that affected the ability of the model to simulate with efficiency the specimen response in the last stages of testing was the re-evaluation of the effective height of the masonry piers. The proper definition of the geometry of spandrels and piers was accomplished by taking into account the developed crack pattern. In general, the crack pattern is not known *a priori*, so this refinement would not be possible in a blind prediction study, where the definition of the effective height should be chosen based on expert judgment or existing proposals.

Results coming from sensitivity analysis showed that the correct assessment of the variation of the roof stiffness during the test evolution constituted another distinct factor in the well-aimed simulation of the global response, therefore the models accounted for the inelastic response of the roof diaphragm. Reference was also made to the importance of properly modelling the construction details and connections between various structural elements. Finally, the sensitivity of the TREMURI model to the assumed Rayleigh damping parameters suggests that the use of different damping models should be investigated and possibly implemented in the future.

ACKNOWLEDGEMENTS

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REFERENCES

[1] Graziotti, F., Rossi, A., Mandirola, M., Penna, A., Magenes, G. (2016a). "Experimental characterization of calcium-silicate brick masonry for seismic assessment". *Proceedings of 16th International Brick/Block Masonry Conference*, Padua, Italy.

[2] Graziotti, F., Tomassetti, U., Penna, A., Magenes, G. (2016b). "Out-of-plane shaking table tests on URM single leaf and cavity walls". *Engineering Structures*, 125, 455-470, doi:10.1016/j.engstruct.2016.07.011.

[3] Graziotti, F., Tomassetti, U., Rossi, A., Kallioras, S., Mandirola, M., Penna, A., Magenes, G. (2017). "Full scale shaking table test on a URM cavity wall terraced house building". *Proceedings of the 16th World Conference on Earthquake Engineering*, Santiago, Chile.

[4] Lagomarsino, S., Penna, A., Galasco, A., Cattari, S. (2013). "TREMURI program: An equivalent frame model for the nonlinear seismic analysis of masonry buildings". *Journal of Engineering Structures*, 56, 1787-1799.

[5] Penna, A., Lagomarsino, S., Galasco, A. (2014). "A nonlinear macroelement model for the seismic analysis of masonry buildings". *Journal of Earthquake Engineering & Structural Dynamics*, 43(2), 159-179.

[6] Bracchi, S., Galasco, A., Penna, A. (2017). "An improved macroelement model accounting for energy dissipation in in-plane flexural behavior of masonry walls". *Proceedings of the 16th World Conference on Earthquake Engineering*, Santiago, Chile.

[7] Peralta, D., Bracci, J., Hueste, M. (2004). "Seismic behavior of wood diaphragms in pre-1950s unreinforced masonry buildings". *J. Struct. Eng.*, 130(12), 2040-2050.

[8] Brignola, A., Pampanin, S., Podesta, S. (2009). "Evaluation and control of the in-plane stiffness of timber floors for the performance-based retrofit of URM buildings". *Bulletin of the New Zealand Society for Earthquake Engineering*, 42(3), 204-221.

[9] Wilson, A., Quenneville, P., Ingham, J. (2013). "In-plane orthotropic behavior of timber floor diaphragms in unreinforced masonry buildings". *J. Struct. Eng.*, 140(1), Article n. 04013038.

[10] Penna, A., Senaldi, I., Galasco, A., Magenes, G. (2015). "Numerical simulation of shaking table tests on full-scale stone masonry buildings". *International Journal of Architectural Heritage: Conservation, Analysis, and Restoration*, 10(2-3), 146-163.

[11] Priestley, M.J.N., Grant, D.N. (2005). "Viscous damping in seismic design and analysis". *Journal of Earthquake Engineering*, 9(2), 229-255.