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A STUDY ABOUT OPTIMAL STIFFENING OF TIMBER FLOORS IN URM BUILDINGS

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ABSTRACT

Timber floors in traditional masonry buildings normally have limited in-plane stiffness, which may be not sufficient to avoid out-of-plane failure of walls or to transmit efficiently seismic forces among walls. Therefore, various stiffening techniques of timber floors have been developed with the aim of improving the global behaviour of the building. The evaluation of the efficiency of the stiffening intervention needs adequate numerical modelling strategy, taking into account the nonlinear in-plane behaviour of masonry piers and spandrels, the out-of-plane stiffness and strength of walls, the actual stiffness and hysteretic behaviour of timber floors. The macro-element modelling can be considered an intermediate strategy in terms of model complexity, as it requires experimental data for its calibration, but can be quite easily adapted to the building geometry. Nonlinear incremental dynamic analyses of different case-study buildings are presented, varying the type of floor, the seismic signal and the modelling criteria as the complexity and accuracy of the adopted technique, with the aim of analysing the effects of the stiffening techniques on the building response. The comparative analyses show that the seismic capacity of a traditional masonry building may decrease if a retrofitting method leading to excessive floor stiffening and/or mass increase is adopted, depending on the geometry and mechanical characteristics of walls and floors. This means that the need of increasing the in-plane stiffness of floors should be evaluated on a case-by-case basis, comparing the actual capacities of floors and walls.

Keywords: URM buildings; timber floors; seismic behaviour; stiffening techniques; nonlinear modelling

1. INTRODUCTION

The evaluation of the actual seismic behaviour of unreinforced masonry (URM) buildings with flexible diaphragms is a matter of constant concern, both at research and professional level, especially when seismic retrofitting interventions have to be designed. However, the numerical modelling of such building typology requires the assumption of hypotheses, simplifications and parameters, relative to masonry and floors, which inevitability leads to uncertainty in predicting the effectiveness of the intervention and the modified response of the building. Moreover, no specifications defining minimum level of detail and the most suitable type of analysis are normally available.

The most comprehensive modelling strategy consists in including each structural component with its actual behaviour, i.e.: the non-linear in-plane behaviour of masonry piers and spandrels; the out-of-plane stiffness and strength of walls; the actual stiffness and hysteretic behaviour of stiffened or unstiffened timber floors. The numerical effort that this requires, led to the development of alternative strategies to simulate the behaviour of URM buildings, with different level of complexity. A first strategy consists in the use of detailed nonlinear Finite Element (FE) models. This method provides an accurate description of the structure but requires high computational effort and uncertainty in evaluating displacement limits (Calderini et al. 2009). A strong simplification of this approach, consists in neglecting the possibility of sliding and diagonal-cracking failures for masonry and the stiffening effects

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of spandrels, adopting fibre beam models (Spacone et al. 1996) for masonry piers. Such simplified modelling technique could be justified if spandrels are weak and therefore masonry structural system can be schematized as cantilever piers, for which only rocking behaviour needs to be simulated. Preliminary analytical calculations are needed to verify that rocking failure anticipates other in-plane failures. Two intermediate strategies in terms of model complexity and reliable interpretation of structural behaviour are available in between the detailed and the simplified approach: the equivalent frame model (Magenes 2000, Roca et al. 2005) and the plane macro-element model (Caliò et al. 2012). In the first strategy, each wall is subdivided into nonlinear deformable masonry panels (piers and spandrels) and rigid portions, which connect the deformable ones (Lagomarsino et al. 2013). This approach is suitable for the analysis of complex buildings with reasonable effort but may present some issues when walls do not have regular openings (Calderini et al. 2009). The second approach consists in the discretization of the building using plane macro-elements, able to reproduce the deformation and failure of masonry walls. This strategy requires experimental data for its calibration and acceptable computational effort but its strength relies in an easier adaptability to different building geometries and a straightforward interpretation of results. Recently, the spatial extension of the plane macro elements in three-dimensional macro elements has been proposed and validated to simulate the combined in-plane and out-of-plane behaviour of URM walls (Pantò et al. 2017).
Independently from the chosen modelling strategy, the choice of parameters and in particular of stiffness values plays a critical role in the modelling phase. Specifically, three important issues, when not neglected, are to be addressed in the aforementioned modelling techniques: (a) the correct evaluation of the in-plane behaviour of spandrels; (b) the estimate of the out-of-plane stiffness and strength of walls; (c) the correct evaluation and modelling of the elastic stiffness and hysteretic behaviour of flexible floors. Experimental research has been conducted and results may be useful to calibrate the model or to estimate possible stiffness parameters. For example: (a) Foraboschi (2009), Beyer (2012), Gattesco et al. (2016) and Rinaldin et al. (2017) performed tests to understand the coupling behaviour of piers and spandrels; (b) Simsir et al. (2004), Vaculik and Griffith (2007), Vaculik et al. (2008), Tondelli et al. (2016) analysed the out-of-plane behaviour of walls; (c) Piazza et al. (2008), Valluzzi et al. (2010), Gattesco and Macorini (2014) and Giongo et al. (2015) studied the in-plane behaviour of unstiffened or stiffened timber floors and their force-displacement response. With reference to timber floors, it is worth noting that comparative analyses of test results highlighted the difficulty in generalizing the elastic stiffness of flexible diaphragms, which is of utmost importance in evaluating the actual behaviour of the building. Finally, various tests and models have been performed to evaluate the seismic response of entire URM buildings adopting flexible or stiffened floors (Paquette and Bruneau 2003, Brignola et al. 2008, Betti et al. 2014, Cattari et al. 2015, Nakamura et al. 2017). Stiffening interventions, as stiff and heavy RC diaphragms, have demonstrated not to improve significantly the seismic behaviour of the building. On the contrary, in some cases they result to be inadequate and unfavourable (Piazza et al. 2008, Gattesco and Macorini 2014, Ongaretto et al. 2016, Masi et al. 2016).
In this work, results from subsequent nonlinear numerical analyses of different case-study buildings, which exploit a macro-element model approach, are presented. The nonlinear in-plane behaviour of horizontal diaphragms and vertical walls was implemented in the model and calibrated according to results from cyclic-loading tests of full-scale specimens available in literature. Then, incremental time-history analyses were performed varying the type of floor, the seismic signal and the modelling hypotheses and simplifications, as the complexity and accuracy of the adopted modelling technique. Obtained results allow to compare the effects of the applied stiffening techniques and modelling choices. In detail, the type of failure and the corresponding Peak Ground Acceleration (PGA) have been evaluated for each analysis. The comparative analyses show that the seismic capacity of a traditional URM building may decrease if a retrofitting method leading to excessive floor stiffening and/or mass increase is adopted, depending on the geometry and mechanical characteristics of the building. This means that the need of increasing the in-plane stiffness of floors should be evaluated comparing the actual capacities of floors and walls.

2. NUMERICAL MODELS

The numerical study summarized in this work analysed four case-study buildings, varying the numerical strategy and relative hypotheses, according to the different geometrical complexity. The common approach adopted for all the three-dimensional models was to perform incremental dynamic analyses (IDAs) with increasing PGA up to the near-collapse PGA (PGA_u). The first case study was analysed with the FE software MidasGEN adopting a fibre beam model to simulate the in-plane rocking behaviour of piers. Hence, this basic case-study implied important modelling simplifications, justified by the simple geometry of the building. Preliminary calculations of the load-bearing capacity of the masonry pier according to Magenes and Calvi (1997) confirmed that, for the chosen pier geometry and vertical load, rocking failure anticipates always other in-plane failures. Despite its simplicity, this modelling approach was still capable of providing preliminary interesting outcomes, being a strategy accessible also in professional engineering design. The increasing complexity of the other three case studies required the use of a more refined macro-element model developed with a research-oriented numerical code (OpenSees, McKenna et al. 2000) to reproduce faithfully the actual behaviour of buildings. It is obvious that the computational demand increases with the number of degrees of freedom and nonlinearities included in the model. Therefore, according to typical parametric approaches, the higher was the model detail and complexity, the lower was the number of studied configurations per case study (i.e., the number of variables introduced).

2.1 Case-study buildings and modelling assumptions

Table 1 summarizes for each case study the geometry, the investigated configurations, the variables in the parametric study and the resulting total number of analyses at each PGA level. The first case study (CS-A) has a rectangular plan with dimensions equal to 10.0x8.0m. Here and hereafter the first number refers to the side of the building orthogonal to the imposed earthquake direction. Three configurations were analysed. The first configuration (A-1) is regular in plan, with four masonry piers, having base dimensions equal to 2.0x0.35m. The second configuration (A-2) is irregular in plan; it has a geometry similar to A-1 but walls in the east façade have base dimensions of 1.0x0.35m. The last configuration (A-3) is the same of A-1 with two additional masonry piers in the middle of the floor span, i.e., it is characterized by redundancy of walls in the direction of the seismic action. The second case study (CS-B) is composed of three masonry walls as A-3. The building is regular in plan, with five masonry piers for each wall, having base dimensions equal to 1.50x0.38m. The total plan dimensions are 10.0x13.5m. The third case study (CS-C) has the same geometrical configuration of the 2:3 scaled building analysed in (Brignola et al. 2008) but real dimensions. It is irregular in plan with dimensions equal to 4.0x4.8m and wall thickness equal to 0.38m. Contrary to CS-A and CS-B, the out-of-plane stiffness of walls was not neglected. The last case study (CS-D) comprises four configurations, with plan dimensions equal to 5.0x4.5m (D-1 and D-3) and 8.0x4.5m (D-2 and D-4). Both the walls parallel to the earthquake direction have one opening in the middle. In two configurations (D-1 and D-2) the out-of-plane stiffness of walls was considered; in the other two (D-3 and D-4) it was neglected. Only in CS-D a parameter further analysed was the thickness of walls (parallel and perpendicular to the earthquake direction) assumed equal to 0.25, 0.32 and 0.38m. All the case studies have two storeys and an inter-storey height equal to 3.0m with the exception of CS-D, having only one storey.

The considered stiffening techniques applied to the original timber diaphragms (NS in the follow identifies the non-stiffened floor) are: TB) addition of a layer of timber boards at an angle of 45° fastened with screws to existing beams, SP) use of light-gauge steel plates at an angle of 45°, RC) addition of a lightweight reinforced-concrete slab connected to the timber beams by means of steel rods. More details are available in (Piazza et al. 2008, Baldessari 2010). Due to the similar behaviour shown by buildings stiffened with TB and SP techniques in CS1, SP option was not investigated in some case studies.

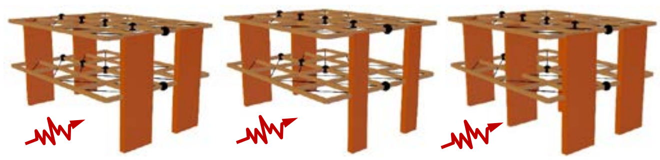
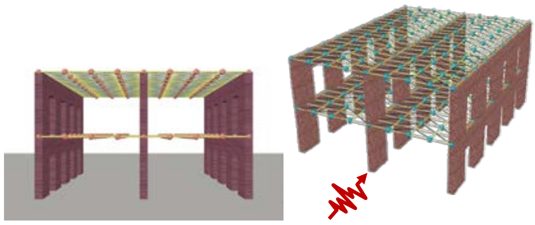
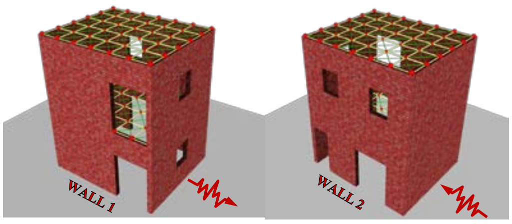
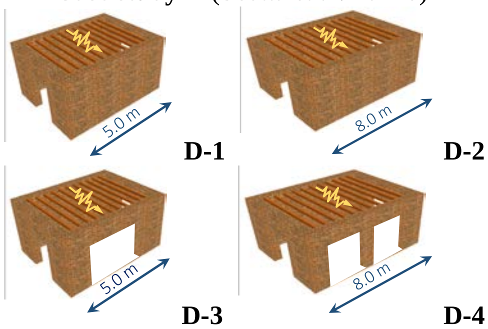
The analysed models have a number of common assumptions, which are independent from their complexity:

- Rigid connections between floors and walls to transmit seismic forces from the horizontal diaphragms to the vertical elements without relative displacements;
- Modelling of the in-plane hysteretic behaviour of floors according to experimental tests, in which loads were applied to the timber beams, in the direction of their axis;

– Absence of a lightweight screed or finishing materials that might alter the in-plane shear stiffness of floors. This implies the use of screeds with loose materials;
– Assumed scale factor of the experimental stiffness of floors according to their plan dimensions;
– Arrangement of translational point masses in the floor nodes according to the relative afferent areas, wall mass lumped at floor levels too;
– Mass and vertical loads computed according to the seismic combination of EN 1990 (2010) assuming live loads equal to 2.00kN/m^2;
– Mass of walls perpendicular to the earthquake direction always considered, even in the case studies in which their stiffness was neglected.

Table 2 lists all the modelling choices, evidencing the differences among the various case studies.

Table 1. Case-study buildings, geometry and configurations

Case studies	Building characteristics and studied configurations (* *variables in the parametric study*)	
Case study A (Scotta et al. 2017a)  A-1 A-2 A-3	Plan dimensions:	10.0 x 8.0 m
	Wall thickness:	0.35 m
	Floor types*:	NS, TB, SP, RC
	Seismic input*:	6 artificial, 1 natural earthquakes
	Pier configurations*:	**A-1** regular (2 walls) **A-2** irregular (2 walls) **A-3** regular (3 walls)
	Total number of configurations:	3 config. x 4 floors
	Total analyses:	84 per PGA level
Case study B (Scotta et al. 2016)  View 1 View 2	Plan dimensions:	10.0 x 13.5 m
	Wall thickness:	0.38 m
	Floor types*:	NS, TB, RC
	Seismic input*:	7 artificial earthquakes
	Pier configuration:	regular (3 walls)
	Total number of configurations:	1 config. x 3 floors
	Total analyses:	21 per PGA level
Case study C (Trutalli et al. 2017)  View 1 (wall 1) View 2 (wall 2)	Plan dimensions:	4.0 x 4.8 m
	Wall thickness:	0.38 m
	Floor types*:	NS, TB, SP, RC
	Seismic input:	1 artificial earthquake
	Pier configuration:	irregular
	Total number of configurations:	1 config. x 4 floors
	Total analyses:	4 per PGA level
Case study D (Scotta et al. 2017b) 	Plan dimensions*:	**D-1 and D-3** 5.0 x 4.5 m **D-2 and D-4** 8.0 x 4.5 m
	Wall thickness*:	0.25 / 0.32 / 0.38 m
	Floor types*:	NS, TB, RC
	Seismic input*:	3 artificial earthquakes
	Pier configuration:	regular
	Total number of configurations:	12 config. x 3 floors
	Total analyses:	108 per PGA level

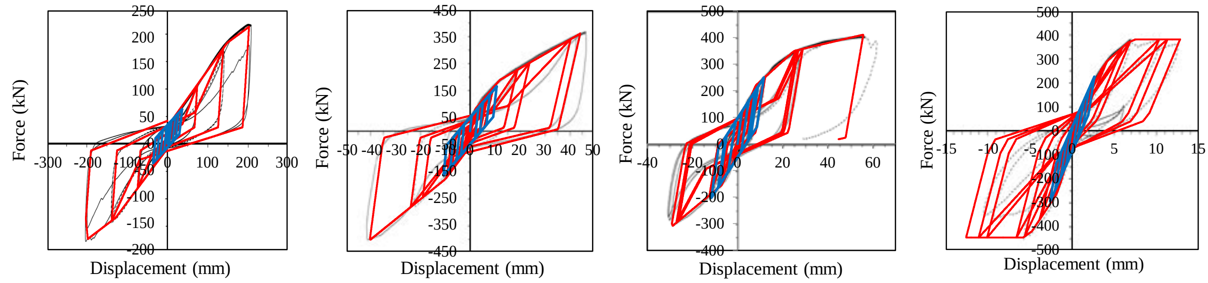
In CS-A, masonry piers were modelled according to a cantilever static scheme, exploiting a fibre beam model capable of representing rocking failure only, typical of slender piers. Another difference between CS-A and the other case studies, is the absence of steel ties at spandrel levels, thus neglecting the coupling effects induced by spandrels. In CS-B, -C and -D, a macro-element approach was used to consider all possible failure mechanisms of masonry piers subjected to lateral loads (i.e., rocking, sliding and diagonal cracking). In these cases, steel ties were included to allow spandrels to work properly in their plane, contributing to the load-bearing capacity of the whole structure, independently from the type of floor. They were modelled as equivalent external compression forces at spandrel levels (CS-B) or truss elements with actual tensile stiffness (CS-C and -D), The out-of-plane stiffness of walls, when considered (i.e., in CS-C and in configurations D-1 and D-2 of CS-D), was modelled by means of a mesh of horizontal and vertical beams with negligible axial stiffness and a reduced effective bending stiffness.
With reference to floor modelling, a nonlinear macro-element approach was exploited to evaluate the inelastic response of the considered floors. CS-A involved a simplification of the behaviour, considering a bilinear elastic-hardening hysteretic model, which is able to fit well only low-displacement cycles (blue curves in Figure 1). However, the validity of this assumption was checked in the post-processing phase, verifying that the displacement range of floors did not exceed the range considered in the calibration, up to failure of the building (failure of masonry in all the analyses). On the contrary, the more refined models of CS-B, -C and -D allowed to replicate faithfully the actual hysteretic behaviour of floors up to their failure, with a multilinear hysteretic model (Lowes and Altoontash 2003) able to simulate also the pinching effect typical of timber structures (red curves in Figure 1).

Table 2. Resume of main modelling choices for each numerical model

Case study	CS-A	CS-B	CS-C	CS-D
Numerical code	Midas Gen	OpenSees	OpenSees	OpenSees
Masonry model	Fibre beams	Macro elements	Macro elements	Macro elements
Failures of masonry	Rocking	All	All	All
Effects of spandrels	No	Yes	Yes	Yes
Out-of-plane stiffness of walls	No	No	Yes	D-1 Yes D-2 Yes D-3 No D-4 No
Irregularity in plan	A-1 No A-2 Yes A-3 No	No	Yes	No
Floor model	Bilinear	Multilinear with pinching	Multilinear with pinching	Multilinear with pinching

2.2 Calibration of floor models

Diaphragms were modelled according to a phenomenological approach, subdividing the floor dimensions into modules composed of stiff elastic truss elements at the perimeter and nonlinear hysteretic behaviour concentrated exclusively in the equivalent diagonals. This modelling strategy has already been used in literature to simulate the nonlinear behaviour of deformable timber shear walls (e.g., Pozza et al. 2015). The dimensions of the macro elements were chosen to fit the discretization used in the wall modelling and to distribute the floor masses. The calibration was performed reproducing the quasi-static tests performed at the University of Trento by Piazza et al. (2008), Figure 1. Therefore the same details and mechanical parameters of the tested specimens have been assumed. The non-stiffened floor (NS) is realized with a single layer of 20x3cm C22-class timber planks nailed orthogonally to 18x18cm GL24c-class timber beams having spacing of 50 cm and fastened with 4 nails per intersection. The first stiffening technique (TB) consists of a second layer of 30-mm thick timber boards arranged at an angle of 45° to the first plank and fastened to the beams with 6x90-mm structural timber screws (from 2 to 4 screws per intersection). The second adopted method (SP) consists of the addition of light-gauge steel plates (80x2mm) screwed to the first boards at an angle of 45° with 5x25mm screws (20 screws per meter). Spacing of diagonal bracing plates is 705mm. The last chosen technique (RC) assumes the addition of a 50-mm thick RC slab reinforced with 6-mm diameter rebars (mesh 200x200mm). Connection between timber beams and RC slab consists of 14-mm diameter L-shaped steel bars spaced 20-30cm glued with epoxy resin.



(a) (b) (c) (d)

Figure 1. Calibration of the hysteretic behaviour of floors. In blue the cycles of CS-1 floors, in red the cycles of the other case studies. (a) NS floor; (b) TB floor; (c) SP floor; (d) RC floor

2.3 Calibration of masonry walls

The fibre beams of CS-A, representing the flexural behaviour of piers and rocking failure, were calibrated using a nonlinear damage model in compression and limited strength in tension, according to Kent and Park model (1971), extended by Scott et al. (1982) and Scotta et al. (2009). Damaging of masonry based on increasing cyclic deformation was considered according the equations proposed by Karsan and Jirsa (1969). The mechanical parameters were calibrated according to tests available in literature (Kaushik et al. 2007).
For CS-B, -C and -D, the in-plane behaviour of masonry walls (piers and spandrels) was reproduced with plane macro elements, each one consisting of an articulated quadrilateral made of rigid elastic trusses, and a series of non-linear springs to simulate all possible in-plane failures and the interaction among elements. Figure 2 shows as example a detail of masonry model of CS-B and the full model of CS-D-1 buildings.

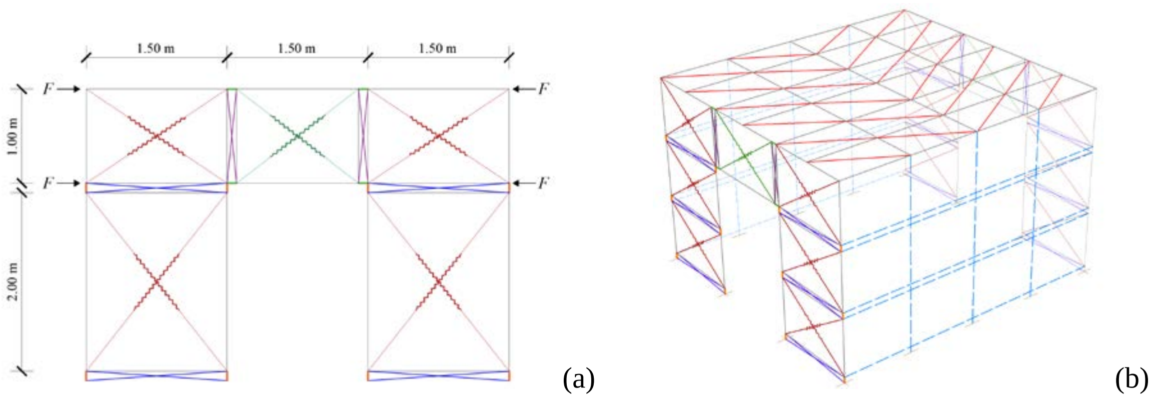


Figure 2. (a) Detail of CS-B model; (b) Full model of CS-D-1 with masonry macro elements, out-of-plane beams and floor macro elements

The following constitutive models have been used:
- Elastoplastic law with softening and failure imposed at an assigned strain limit to simulate diagonal-cracking behaviour, depending on the vertical load on the panel, according to Turnšek and Čačovič (1970) criterion;
- Compression-resistant elastic-plastic law with limited tensile strength to simulate rocking failure;
- Symmetrical elastic-perfectly plastic law with failure at a given strain to simulate shear-sliding failure, accounting for vertical load according to Mohr-Coulomb criterion.

The in-plane behaviour was calibrated according to experimental tests of URM walls available in literature (Magenes and Calvi 1992). Figure 3 shows the numerical simulation of a test of a masonry pier performed by Magenes and Calvi (1992) and of a wall with an opening performed by Allen et al. (2015).

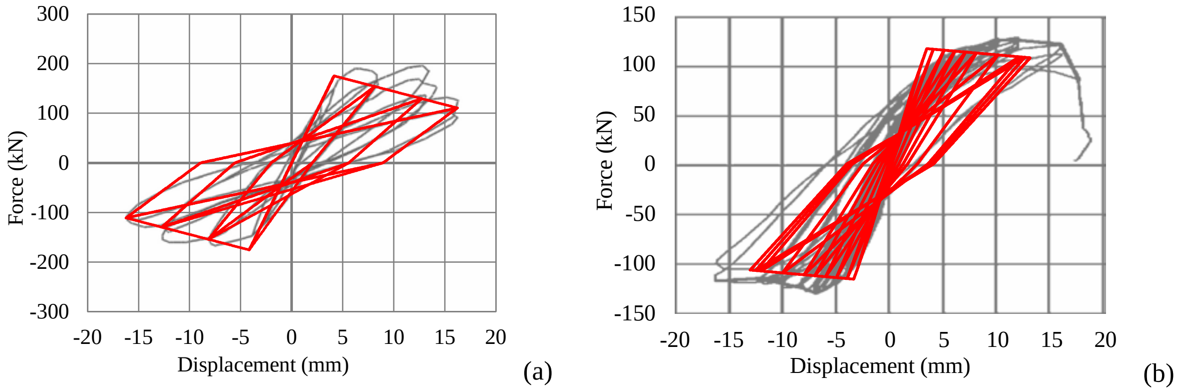


Figure 3. Numerical simulation (red curves) of experimental tests (grey curves): (a) Test of a masonry pier (Magenes and Calvi 1992); (b) Test of a masonry wall with an opening (Allen et al. 2015)

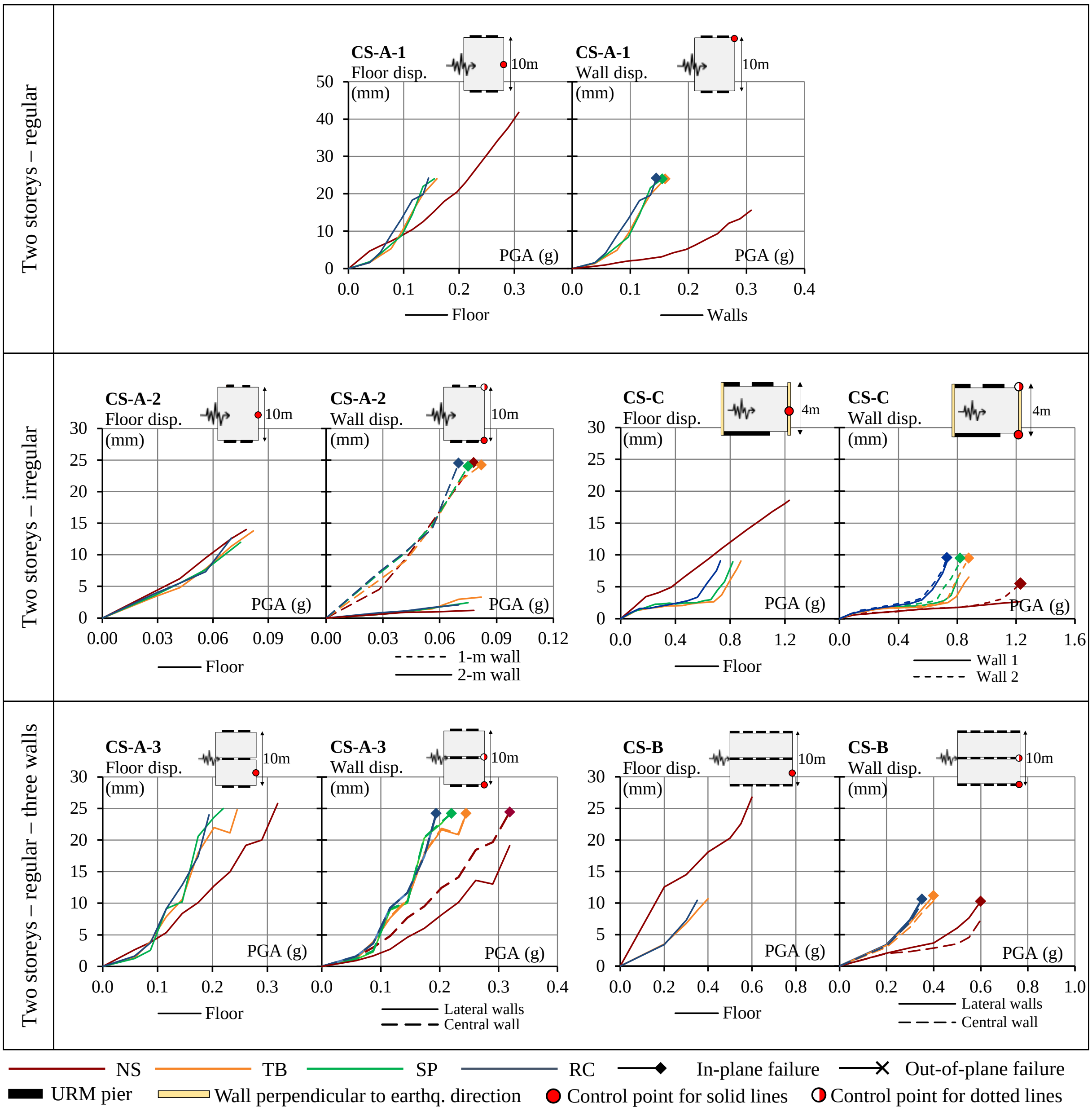
3. DISCUSSION OF RESULTS

IDAs were performed considering a different number of earthquake signals for each case study, according to Table 1. Artificial earthquakes (Gelfi 2012) were generated respecting the spectrum compatibility requirements according to the elastic response spectrum (CEN 2013) for building foundations resting on type A soil, q-factor=1, maximum spectral amplification factor F_0 equal to 2.5 and building importance factor γ_I=1. The PGA was increased by applying small-amplitude steps up to the near-collapse condition at PGA_u, defined at the achievement of one of the following failures: in-plane failure of walls, out-of-plane failure of walls, or in-plane failure of floors. In-plane failure condition for walls was assumed depending on masonry modelling. For CS-A, it was defined at the achievement of ultimate deformation capacity of compressed masonry, which corresponded to an inter-storey drift equal to 0.8%. For the other case studies, it was defined at the first achievement of the maximum deformation capacity by a nonlinear element. The out-of-plane failure of the walls orthogonal to the seismic input was defined as the achievement of an absolute ultimate inter-storey drift equal to 2.0% at mid-span floor. Finally, the possible failure of floors was accounted assuming a maximum deformation capacity of the nonlinear truss elements. Actually, such limit was never reached in the various analyses.

Main outcomes from the seventeen configurations are summarized in Figure 4, which shows displacement *vs.* PGA graphs, up to in-plane or out-of-plane failure of walls, specified with a different marker. For CS-A and CS-B, displacements for each PGA level are averaged among the seven earthquakes applied; results of CS-C refer to the unique seismic signal, whereas for CS-D are referred to the worst seismic signal among the three considered. All graphs show the mid-span displacements of the first floor and displacements of walls at the same level (see the control points in the plan views above each graph).

In all the case studies, only the NS floor (red curves) underwent significant relative displacements, whereas all the trend lines of stiffened floors are close to or overlap the lines of walls, i.e., the stiffened floors are rigid or slightly flexible with reference to their geometry and stiffness of walls. The stiffened floors remained in their elastic range, with the only exceptions of TB floors (orange curves) in CS-D-2, -3 and -4 when 380-mm thick walls were assumed.

Analysing the single case studies some conclusions can be drawn. With reference to CS-A, only in the regular configuration with 10-m wide floors (CS-A-1) the NS floor allowed large relative displacements. Inelastic behaviour of NS floor is responsible of large energy dissipation and reduction of forces transmitted to shear walls. Oppositely, whatever stiffening of floors impaired their dissipative capacity. Despite a strong reduction of relative displacement, such stiffening produced an increase of wall displacements for a given PGA and consequently an impairment of seismic performance of building respect the unstiffened floor. The other two configurations of CS-A led to different conclusions. As expected, the plan irregularity and the reduced strength of a wall of CS-A-2 were responsible for an earlier collapse of the building than CS-A-1, due to rocking failure of the weakest wall. In this case, the stiffness of NS floors, compared to that of the weakest wall, was enough to confer a rigid-diaphragm behaviour with a global torsional distortion of the building. In configuration CS-A-3, with three parallel walls (redundant scheme) the deformability of NS floors was not negligible. Nevertheless, stiffness was enough to assure a quite good distribution of seismic forces among the resisting shear walls. As for CS-A-1, adoption of rigid and non-dissipative floors led to increased displacements of walls for a given PGA and to a reduction of PGA_u.



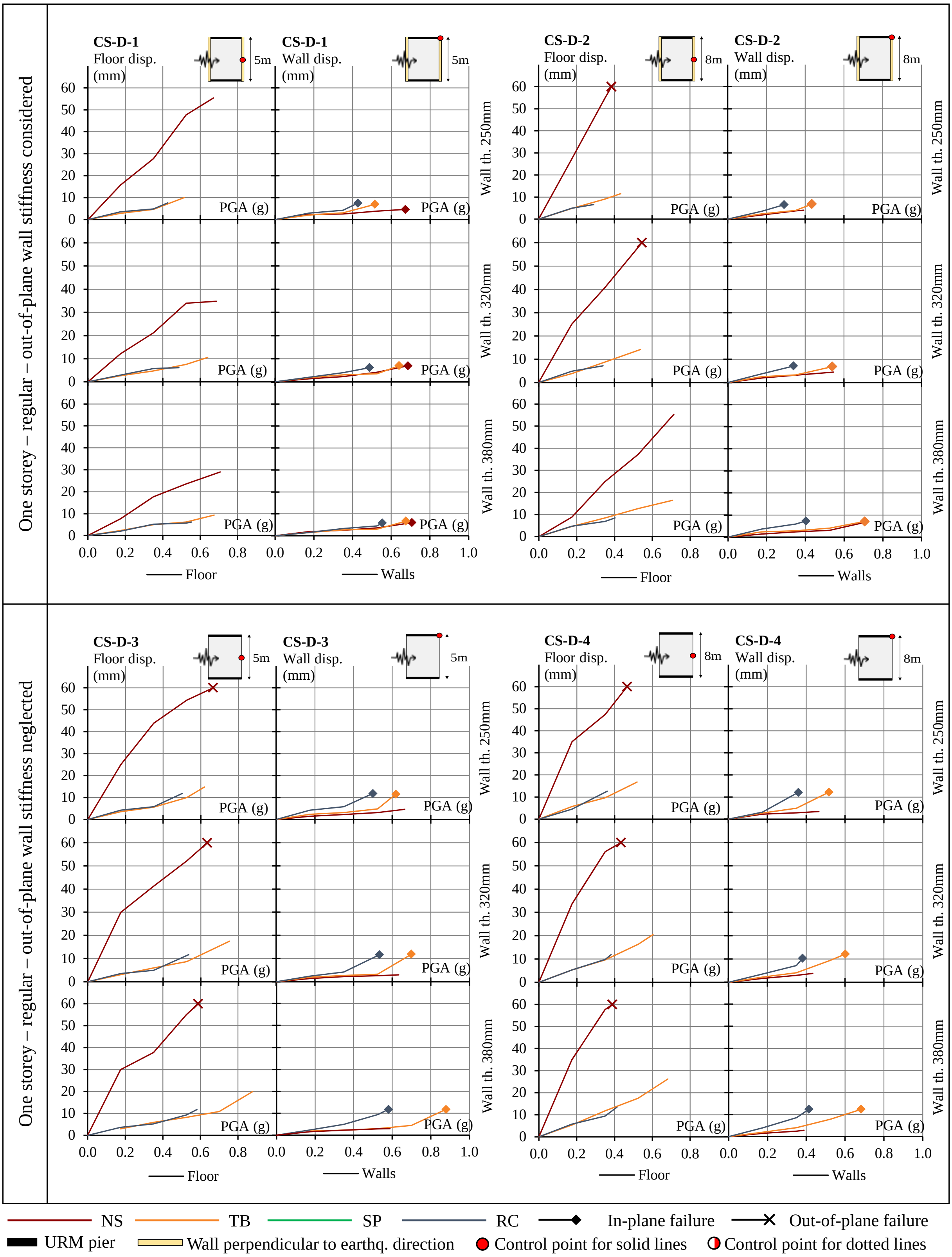


Figure 4. Results from IDAs in terms of displacements of floor and walls at first floor level at increasing PGA levels. Red dots represent the control points. For CS-A and CS-B displacements were averaged over the seven earthquakes. For CS-C results refer to the applied earthquake. For CS-D results of the strongest earthquake among the three seismic signals are plotted; results are presented for each wall thickness (wall th.)

Similar conclusions can be drawn considering the other irregular configuration (CS-C) and redundant configuration (CS-B), for which a different modelling strategy and different hypotheses were assumed (see Section 2.1). The PGA_u was reached in both cases due to diagonal cracking failure localized in the masonry piers of a wall at about 3‰ to 4‰ drift. PGA_u higher than values for CS-A were recorded, consistently to the different strength of walls and/or building dimensions. Results evidence again the effectiveness of the considered stiffening techniques in limiting in-plane deformation of floors and therefore in distributing horizontal forces among walls. However, also in these case studies, a decrease of PGA_u was recorded for the stiffened configurations, due to reduced displacement and dissipative capacities. It is worth emphasizing that in all the configurations analysed hereinbefore, the stiffened diaphragms showed a very similar behaviour, with a slightly worse behaviour for the RC floors, due to the increased seismic mass of the added concrete layer. The parametric study of CS-D shows additional and sometimes different trends, depending on the relation between wall and floor stiffness. Comparing the absolute displacements of walls and floors for CS-D-1 and CS-D-2 buildings, it can be seen that the RC floor can be considered rigid in all cases, whereas TB floor underwent relative displacements, higher for CS-D-2 than CS-D-1 due to increased width of floor. However, the stiffness was sufficient to limit the out-of-plane displacements of walls orthogonal to seismic input. As CS-A, -B and -C, the NS floor underwent significant relative displacements, which in some cases of CS-D-2, led to exceed 2% drift, i.e., the assumed limit of out-of-plane failure of walls. The direct consequence was an earlier failure of the building, which was not able to exploit entirely the in-plane strength capacity of walls. This result is obviously more evident in CS-D-3 and CS-D-4, which have the same plan dimensions of CS-D-1 and CS-D-2 respectively, but do not consider the out-of-plane stiffness of walls, implying higher displacements at mid-span floor. In these cases, the NS floor reached always the collapse due to excessive out-of-plane displacements of walls, whereas the TB floor showed an optimal behaviour, as it combined the advantages of partial stiffening with a minimum mass increase, as opposed to RC floor, which resulted to be too stiff and not effective due to mass increase. Comparing the effects of the TB and RC floor, varying the width of the floor (from 5 to 8m) and maintaining unchanged the strength and stiffness of shear walls, it is evident that the larger are the floor dimensions, the higher is the mass increase due to the added concrete layer in the RC technique, with the consequent effectiveness reduction of this type of intervention.

4. CONCLUSIONS

Nonlinear incremental dynamic analyses of four case-study URM buildings, varying geometry, type of floor, and modelling strategy, have been performed with the aim of analysing the effects of the stiffening techniques on the building response. A first general conclusion is that floor stiffening interventions may not improve the actual seismic performance of the building. On the contrary, excessive floor stiffening and/or mass increase may lead to decrease the seismic performance of the building. If the stiffness of a flexible floor is sufficient to avoid first-mode failures and to guarantee an effective transmission of forces among shear walls, its dissipative capacity reduces the seismic forces transmitted to shear walls. Such interventions require that a correct ratio between the increase of in-plane stiffness of floors and the decrease of their dissipative capacity be defined, with suitable modelling techniques, providing that in the analyses of URM buildings with flexible diaphragms, the out-of-plane displacements of walls orthogonal to the seismic action be always controlled. Finally, it is worth noting that the case-study buildings do not replicate existing URM buildings and might appear too simplistic and not easily applicable in current practice but they are consistent in analysing the actual interaction between the stiffened timber floors and the URM walls. The outcomes of this work derive from numerical models calibrated according to specific tests and are limited to the specific geometries of the buildings. Further research is needed to generalize the results.

5. ACKNOWLEDGMENTS

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