Evaluation of timber floor in-plane retrofitting interventions on the seismic response of masonry structures by DEM analysis: a case study

Alessandra Gubana (✉ alessandra.gubana@uniud.it)

University of Udine: Università degli Studi di Udine <https://orcid.org/0000-0003-3428-5123>

Massimo Melotto

University of Udine: Università degli Studi di Udine

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Abstract

The seismic response of existing masonry structures is strongly influenced by floor and roof in-plane properties. An in-plane strengthening intervention is often needed on traditional timber floors to overcome their low in-plane stiffness and to preserve historical buildings. In this study, the effect of un-stiffened and stiffened timber floors on the seismic behaviour of an existing listed masonry building is investigated with dynamic non-linear analyses by means of the Discrete Element Method (DEM). With this approach, the failure processes and collapse sequences of masonry structures can be followed in detail.

A previously developed model of the floor cyclic behaviour, based on experimental data, is here applied in the DEM models of the masonry building. Different seismic ground accelerations, different floor types and different wall-to-diaphragm connections are considered.

The results highlight the effectiveness of the analysed floor strengthening solution in reducing the out-of-plane displacements of masonry walls. With adequate connections, the reinforced floor is able to transfer the seismic forces to the shear resistant walls up to the shear-sliding collapse of the side walls of the structure. A comparison with the ideal rigid diaphragm case confirms the good performance of the strengthened floors. The small observed out-of-plane displacements are compatible with the masonry wall capacity, and the reinforced floor hysteretic cycles contribute to dissipating part of the input energy. Moreover, different designs of the connections can also cap the transferred seismic forces to an acceptable level for seismic resistant walls.

1. Introduction

The seismic assessment and retrofitting of existing masonry buildings is still a challenging topic in structural engineering due to the complexity and variability of unreinforced masonry structures and to the several factors affecting their dynamic behaviour.

Among these factors, the in-plane flexibility of traditional timber floors and the lack of effective connections to load bearing walls usually cause the development of local collapse mechanisms. Floors and roofs that are stiffer in-plane allow effective transfer of the seismic load to the shear resistant walls and reduction of the out-of-plane overturning of perimeter walls, thus increasing the building seismic performance.

Several in-plane strengthening techniques for timber floors have been experimentally studied in recent years, with attention to the reversibility of the intervention and to its compatibility with the existing parts of the buildings. These solutions may use steel elements, fibre-reinforced polymer strips, timber boards or timber-based panels (Gubana, 2015). Numerical studies and analytical models on the in-plane performance of flexible and strengthened timber floors are described in (Peralta et al. 2003, Whitney et al. 2015, Rizzi et al. 2017, Giongo et al. 2018, Rizzi et al. 2019a, Rizzi et al. 2019b, Metelli et al. 2019, Gubana and Melotto 2019b).

The influence of floor mechanical properties on the global seismic behaviour of masonry buildings has been addressed in different studies using linear dynamic numerical analysis (Tena-Colunga and Abrams, 1996), push-over analysis (Giongo et al. 2012, Gattesco and Macorini 2014, Ortega et al. 2018, Jiménez-Pacheco et al. 2020) or non-linear dynamic analysis (Betti et al. 2014, Gubana and Melotto 2017, Nakamura et al. 2017, Scotta et al. 2018, Gubana and Melotto 2019a, Trutalli et al. 2019). The possibility of designing and calibrating the in-plane properties of the floor to cap the shear forces addressed to the shear resistant walls has also been investigated in previous studies (Preti et al. 2017, Longarini et al. 2019).

Other than the in-plane floor properties, the quality of the connections between the floors and vertical elements strongly influences the seismic response. Proper connections are needed to reduce the vulnerability to out-of-plane actions; however, in most existing masonry buildings, the link between timber beams and walls is mainly friction-based. In many cases, timber beams are simply supported by perimeter walls inserted in pockets with no embedment. Many solutions have been used and studied to connect joists to masonry walls by using steel elements anchored to the floor. A review of the different techniques can be found in Moreira et al. (2014).

A study of the influence of the floor in-plane hysteretic properties and the floor-to-walls connection properties on the seismic response of a typical masonry heritage building is reported here.

The analysis is conducted by means of the Discrete Element Method (DEM), recently applied to masonry structures, as it allows to consider the complete separation of bodies and the formation of new contacts during the evolution of the seismic event. Stresses and deformations are transmitted by contact forces between blocks and failure processes, and collapse sequences can be followed in detail. Therefore, this approach can be adopted to better understand the complex dynamic behaviour of masonry structures under seismic action (Lancioni et al. 2012, Lemos and Campos Costa 2017, Bui et al. 2017) and to simulate all the mechanisms observed in masonry buildings without “box behaviour” (out-of-plane rocking and out-of-plane collapse of masonry piers). Moreover, recent studies (Baraldi et al., 2018, Bui et al., 2019, Baraldi et al. 2020, Pulatsu et al. 2020) confirm the efficiency and robustness of the DEM to simulate the in-plane behaviour of regular masonry wall panels.

Gubana and Melotto (2019a) first investigated a simple masonry cell via DEM due to the high computational cost of this analysis type. The results of this work pointed out the effectiveness of the considered strengthening interventions in preventing the triggering of out-of-plane mechanisms of masonry walls.

In this paper, a more complex structure is addressed, and a heritage building is considered as a case study. The dynamic response of the structure with unstrengthened and strengthened floors is compared. The effect of the actual cyclic hysteretic response of the floor and its capability to dissipate energy are investigated by using a specifically developed floor model combined with the DEM masonry model.

The cyclic behaviour assigned to either unreinforced or reinforced floors is derived from previous tests by the authors (Gubana and Melotto 2018). The experimental samples replicate traditional timber floors unreinforced and reinforced with timber-based panels connected to the original floor by means of nails or self-tapping screws. These techniques are reversible and minimally invasive and are characterised by a small mass and low thickness.

Five different earthquake records are applied for the analyses. Three of these records are scaled to be compatible with the local spectrum. The other two are the original records and are characterised by greater magnitudes. The results are compared, considering the out-of-plane displacement of the masonry walls, the base reaction of the structures, the dissipated energy and the collapse mechanisms.

In Sect. 2, the building used as a case study is described. In Sect. 3, the modelling strategies for masonry walls and timber floors are reported. In Sect. 4, the numerical results are reported for the five seismic records.

2. Description Of The Case Study

The building, selected as a case study, is a typical example of a noble villa in north-eastern Italy. The current appearance is due to two important renovations of an ancient country residence of the Cattaneo family. One renovation began in 1718, and the second renovation began at the end of the 18th century. At that time, the two-storey palace was raised two floors with respect to the original building, and a new principal façade crowned by a triangular tympanum with a star decoration inside was built substituting the original façade on the orthogonal side. Due to this intervention, the internal wall layout of the new storeys does not match that of the lower storeys, as better explained in the following. The 4th storey covers only a small portion of the building.

The main façade of the villa (south) and the west façade are shown in Fig. 1.

The building has a compact rectangular shape of approximately 16 m by 15 m in size.

The drawings of the floor plans, of the section of the building and of the front façade are shown in Fig. 2. The internal layout at each storey is characterised by a central hall, long as the building, with rooms on its two sides. The internal walls are mainly oriented in the east-west direction, and the timber floor beams are oriented in the north-south direction. The walls are made of rubble stone masonry and have thicknesses from 55 cm (ground floor) to 45 cm (upper storeys).

Non-invasive surveys were carried out using micro cameras inside the depth of the walls and by means of thermographic imaging. The texture and the quality of the masonry allowed to use the reference values suggested in the Italian building code (CS.LL.PP. 2018). Additional visual inspections and ground penetrating radar investigations were also performed to acquire an accurate knowledge of the timber structures (Riggio et al. 2018).

On the ground and first floors, a masonry wall oriented in the north-south direction was added in recent years after the Friuli 1976 earthquake. On the third floor, the main hall is oriented orthogonally to the halls of the lower storeys, so the internal walls have large openings and are positioned only in the east-west direction. This increases the vulnerability of the main façade for out-of-plane mechanisms and makes this building an interesting case study for the evaluation of the floor behaviour effect on the structure dynamic response.

3. Description Of The Numerical Model

The building is selected as a reference case to conduct this study on the effect of timber floor behaviour and thus on the effectiveness of the strengthening solutions. The geometry, the thickness of the walls and the applied loads on the numerical model comply with the actual building. However, the shape of the building is slightly simplified for ease of modelling. Moreover, the roof structure and the small 4th storey are not considered in the model, but their masses are applied to the last floor.

The numerical simulations are carried out by using the DEM with the commercial general-purpose software ABAQUS/Explicit (Abaqus 2011). Figure 3 shows a view of the numerical model. The highlighted points are the control points for the results in the following sections.

3.1 Masonry walls modelling

Rubble stone masonry characterises the walls, but regularly shaped stones are used at the corners. In the DEM, the discontinuities of the masonry are explicitly taken into account by considering masonry blocks that interact through contact points at the interfaces.

The masonry walls are divided into distinct blocks, whose size is approximately 1.0 m x 0.5 m x 0.5 m. The block division is not intended to describe the wall texture. Due to computational limits, the possibility of considering single stone pieces is reserved for isolated masonry walls or very simple structures. However, the chosen block size is considered to be small enough with respect to the building size for the study of collapse mechanisms.

The material assigned to the masonry blocks is isotropic, homogeneous and elastic. The density and elastic modulus are typical of Italian rubble stone masonry, as the one of the considered building (Table 1).

The masonry non-linearity is considered at the interfaces between blocks. In the analyses, a frequently used approach is followed (Lemos and Campos Costa 2017, Bui et al. 2019), where the interaction in the normal direction is of rigid contact with infinite compressive strength. In the tangential direction, a Coulomb isotropic friction relationship and a cohesive model with a tensile-shear Rankine failure criterion are considered. Thus, masonry elements fully adhere at the beginning of the analysis; when the interface failure criterion is reached, the blocks separate, and large displacements can occur. The properties assigned to the blocks and to the interactions are reported in Table 1. The tensile and shear strengths and

the friction coefficient are typical of the considered masonry type. The strength values are those reported in the Italian building code (CS.LL.PP. 2018).

The ABAQUS “General Contact method” is used, and the significant contact pairs are automatically generated by a Python script and are applied as “surface-to-surface contacts”.

Due to the recognised importance of the density of contact points for a correct evaluation of the stress distribution and hence of the failure mechanism, a sensitivity analysis is performed to assess the reliability of the assigned contact model. The contact point density is progressively increased, and a mesh size of 10 cm is considered a reasonable compromise between accuracy and computational time (Gubana and Melotto 2019a) and for this reason is adopted in the present work.

Table 1
Properties assigned to the masonry
blocks and interfaces

Masonry property	Value
Density	2100 kg/m ³
Elastic modulus	1500 MPa
Shear modulus	500 MPa
Friction coefficient	0.7
Tensile strength	0.071 MPa
Shear strength	0.047 MPa

3.2 Timber floor modelling

In the numerical DEM model, accurate modelling of the floor is implemented. The floor is built by timber beams spaced 50 cm, which are inserted in the pockets of the bearing walls. The floor global behaviour is modelled by using non-linear springs between adjacent joists, as shown in Fig. 4 (Gubana and Melotto 2019a). This global model of the floor is properly developed and checked on the basis of the experimental tests in Gubana and Melotto (2018). The spring hysteretic properties are calibrated to exactly reproduce the tested floor results (Gubana and Melotto 2019a). In particular, the assigned properties (stiffness, strength, hardening and damage) are the average properties extrapolated from the experimental results for each floor type.

In those tests, traditional timber floors made by beams and boards are considered unreinforced specimens. The reinforced configurations use an overlay of Oriented Strand Boards (OSB) panels or Cross-Laminated Timber (CLT) panels to increase the in-plane stiffness and strength. Different fasteners (ring-type nails and self-tapping screws) are used. Cyclic tests are performed after loading the samples in the joist direction, and these experiments are replicated in the numerical models.

In the DEM model, the timber beams lay in wall pockets, and a Coulomb friction interaction is considered between the two materials. A friction coefficient $\mu = 0.4$ is chosen by averaging literature data (e.g., Lin and LaFave 2012). The eventual contact between the joist heads and the masonry is taken into account.

The effect of the connection between the floor and the walls is also investigated, as the out-of-plane failure mechanisms in masonry buildings are often a direct result of poor connections between the structural elements. In this research work, three basic assumptions are made: the absence of effective connections for the unreinforced floor case and the presence of elastic connections (stiffness 15 kN/mm) or elastic-plastic connections (stiffness 15 kN/mm, strength 4.0 kN) for the reinforced and rigid floor cases. The properties are chosen considering a 16 mm diameter steel bar embedded in the masonry using the equations reported in Brignola et al. (2012).

3.3 Applied loads

The structure is loaded in two stages. In the first one, gravity is applied, and the equilibrium state is reached. The considered vertical loads are the self-weight of the masonry and of the floors. A floor live load of 3.0 kN/m² is applied as a distributed mass, and the load combinations are those prescribed by the Eurocode (EN1990 2002). The load is chosen in the hypothesis of a public use of the listed building.

In the second stage, an acceleration history is applied to the rigid base in the north-south direction (see Fig. 3).

Three seismic acceleration histories are chosen using the REXEL tool (Iervolino et al. 2009), which provides a group of scaled seismic records compatible with the site spectrum. The records used are the Gemona (Italy) seismic record of September 15th, 1976 (09:21 UTC), the Mirandola (Italy) seismic record of May 20th, 2012, and the San Felice sul Panaro (Italy) seismic record of May 29th, 2012. The three earthquakes are identified with the labels “GMN”, “MRN” and “SAN”. Figure 5a shows the spectra of the three scaled seismic histories and the Italian building code spectrum for the building site (475-year return period).

Notably, the Friuli 1976 earthquake had limited effects on the case study building due to the strong ground attenuation between the fault and the villa site (Bragato and Slejko, 2005).

Other than the three scaled seismic records, compatible with the site spectrum, two real seismic acceleration histories are considered, without scaling. These are from the Friuli 1976 earthquake, whose epicentre was just 50 km away from the villa considered as a case study. The two records are the Gemona (Italy) seismic record of May 6th, 1976 (20:00 UTC) and the Gemona (Italy) seismic record of September 15th, 1976 (03:15 UTC), which are named “May76” and “Sep76” in the following. Figure 5b shows the spectra of these two unscaled seismic histories.

3.4 Summary of the studied cases

In Table 2, a summary of the analysed numerical models is reported. The label assigned to each model identifies the seismic acceleration history (using the strings already defined), namely, the floor type (UR for the unreinforced and unconnected floor; CLT for the floor reinforced by means of CLT panels; RIG for the ideal rigid case) and the floor-to-wall connection type (elastic, EL, or elastic-plastic, EP).

Table 2
Summary of the analysed models

Label	Earthquake	Floor type	Floor-to-walls connection
GMN-UR	Gemona (Italy) seismic record of September 15th, 1976 – scaled to the site spectrum	Unreinforced	None
GMN-CLT-EL		Reinforced	Elastic
GMN-CLT-EP		Reinforced	Elastic-plastic
GMN-RIG-EL		Rigid	Elastic
MRN-UR	Mirandola (Italy) seismic record of the May 20th, 2012 – scaled to the site spectrum	Unreinforced	None
MRN-CLT-EL		Reinforced	Elastic
MRN-CLT-EP		Reinforced	Elastic-plastic
MRN-RIG-EL		Rigid	Elastic
SAN-UR	San Felice sul Panaro (Italy) seismic record of the May 29th, 2012 – scaled to the site spectrum	Unreinforced	None
SAN-CLT-EL		Reinforced	Elastic
SAN-CLT-EP		Reinforced	Elastic-plastic
SAN-RIG-EL		Rigid	Elastic
May76-CLT-EL	Gemona (Italy) seismic record of May 6th, 1976 (20:00UTC) – unscaled	Reinforced	Elastic
May76-CLT-EP		Reinforced	Elastic-plastic
Sep76-CLT-EL	Gemona (Italy) seismic record of September 15th, 1976 (09:21UTC) - unscaled	Reinforced	Elastic
Sep76-CLT-EP		Reinforced	Elastic-plastic

4. Numerical Results

The results of the DEM analysis for the three scaled ground motions are plotted in Fig. 6 to Fig. 8. Several parameters are chosen for the comparison:

- a) the out-of-plane displacements of the masonry walls loaded out-of-plane (south façade, point P1.3 in Fig. 3);
- b) the in-plane displacements of the lateral walls (point P4.3 in Fig. 3);
- c) the base reaction force history;
- d) the energy dissipated over time;
- e) the load transferred from the floor to the lateral walls (elastic or elastic-plastic floor-to-wall connections).

The reported energy values are the kinetic energy of the structure, the energy dissipated by the floor hysteretic behaviour and the energy dissipated by the masonry walls due to damage and friction effects.

All the results are summarised in Table 3 and Table 4. For each seismic record and for each floor type, the first table reports the out-of-plane displacements of the face-loaded masonry walls and the corresponding inter-storey drift. The second table reports the in-plane displacements of the lateral walls and the minimum and maximum base reaction force values. The displacements are measured at each floor level, and the minimum and maximum values are reported.

Table 3

Earth-quake	Floor type	Height [m]	Out-of-plane displacement P1 [m]		Inter-storey drift south façade [%]		Out-of-plane displacement P2 [m]		Inter-storey drift north façade [%]	
			max	min	max	min	max	min	max	min
GMN	UR	3.0	0.022	-0.001	0.73	0.03	0.004	-0.024	0.13	0.80
		6.5	0.071	-0.008	1.40	0.20	0.028	-0.090	0.69	1.89
		11.0	0.105	-0.147	0.76	3.09	0.089	-0.171	1.36	1.80
CLT-EL		3.0	0.004	-0.007	0.23	0.003	-0.010	0.10	0.33	
		6.5	0.011	-0.031	0.69	0.011	-0.037	0.23	0.77	
		11.0	0.024	-0.105	1.64	0.024	-0.108	0.29	1.58	
CLT-EP		3.0	0.008	-0.001	0.03	0.004	-0.010	0.13	0.33	
		6.5	0.020	-0.014	0.37	0.017	-0.039	0.37	0.83	
		11.0	0.042	-0.131	2.60	0.040	-0.134	0.51	2.11	
RIG		3.0	0.005	-0.006	0.20	0.003	-0.008	0.10	0.27	
		6.5	0.012	-0.028	0.63	0.012	-0.034	0.26	0.74	
		11.0	0.025	-0.123	2.11	0.025	-0.112	0.29	1.73	
MRN	UR	3.0	0.013	-0.001	0.43	0.03	0.003	-0.019	0.10	0.63
		6.5	0.071	-0.005	1.66	0.11	0.015	-0.054	0.34	1.00
		11.0	0.239	-0.067	3.73	1.38	0.208	-0.091	4.29	0.82
CLT-EL		3.0	0.007	-0.006	0.20	0.006	-0.010	0.20	0.33	
		6.5	0.037	-0.027	0.60	0.034	-0.034	0.80	0.69	
		11.0	0.102	-0.075	1.44	1.07	0.098	1.42	0.53	
CLT-EP		3.0	0.009	-0.002	0.07	0.048	-0.013	1.60	0.43	
		6.5	0.042	-0.014	0.34	0.033	-0.047	-0.43	0.97	
		11.0	0.102	-0.068	1.20	0.098	-0.072	1.44	0.56	
RIG		3.0	0.007	-0.005	0.17	0.006	-0.007	0.20	0.23	
		6.5	0.031	-0.019	0.40	0.030	-0.025	0.69	0.51	
		11.0	0.091	-0.063	0.98	0.088	-0.058	1.29	0.73	
SAN	UR	3.0	0.042	-0.001	1.40	0.03	0.004	-0.035	0.13	1.17
		6.5	0.125	-0.025	2.37	0.69	0.015	-0.132	0.31	2.77
		11.0	collapse	-0.154	collapse	2.87	collapse	-0.182	collapse	1.11
	CLT-EL	3.0	0.009	-0.008	0.30	0.27	0.006	-0.011	0.20	0.37
		6.5	0.041	-0.024	0.91	0.46	0.039	-0.030	0.94	0.54
		11.0	0.090	-0.078	1.09	1.20	0.107	-0.065	1.51	0.78
	CLT-EP	3.0	0.014	-0.005	0.47	0.17	0.003	-0.015	0.10	0.50
		6.5	0.063	-0.014	1.40	0.26	0.045	-0.045	1.20	0.86
		11.0	0.116	-0.071	1.18	1.27	0.116	-0.077	1.58	0.71
	RIG	3.0	0.008	-0.007	0.27	0.23	0.006	-0.010	0.20	0.33
		6.5	0.030	-0.021	0.63	0.40	0.030	-0.025	0.69	0.43
		11.0	0.101	-0.075	1.58	1.20	0.109	-0.069	1.76	0.98

Table 4 Summary of the results of the different numerical models. In-plane displacements and base reactions

Earth-quake	Floor type	Height [m]	In-plane displacement point P3 [m]		In-plane displacement point P4 [m]		Base reaction [kN]	
			max	min	max	min	max	min
GMN	UR	3.0	0.005	-0.001	0.003	-0.003	1737.8	-2227.4
		6.5	0.016	-0.005	0.007	-0.009		
		11.0	0.031	-0.011	0.010	-0.019		
CLT-EL		3.0	0.004	-0.008	0.003	-0.010	2213.5	-1991.4
		6.5	0.010	-0.028	0.008	-0.033		
		11.0	0.021	-0.061	0.013	-0.082		
CLT-EP		3.0	0.005	-0.004	0.004	-0.006	2167.5	-2298.2
		6.5	0.012	-0.013	0.009	-0.021		
		11.0	0.048	-0.028	0.012	-0.048		
RIG		3.0	0.005	-0.006	0.004	-0.009	2250.6	-1909.8
		6.5	0.013	-0.027	0.012	-0.033		
		11.0	0.026	-0.069	0.025	-0.084		
MRN	UR	3.0	0.006	-0.001	0.003	-0.003	1777.0	-2337.5
		6.5	0.015	-0.004	0.008	-0.011		
		11.0	0.035	-0.009	0.013	-0.019		
CLT-EL		3.0	0.008	-0.004	0.006	-0.007	2236.3	-2345.6
		6.5	0.027	-0.013	0.025	-0.020		
		11.0	0.065	-0.014	0.059	-0.039		
CLT-EP		3.0	0.008	-0.003	0.004	-0.006	2192.6	-2278.8
		6.5	0.028	-0.009	0.017	-0.016		
		11.0	0.065	-0.010	0.035	-0.035		
RIG		3.0	0.010	-0.006	0.006	-0.007	2233.7	-2294.5
		6.5	0.030	-0.020	0.028	-0.026		
		11.0	0.093	-0.031	0.077	-0.047		
SAN	UR	3.0	0.008	-0.003	0.005	-0.005	2416.1	-2982.3
		6.5	0.022	-0.009	0.010	-0.014		
		11.0	0.051	-0.010	0.017	-0.027		
	CLT-EL	3.0	0.008	-0.007	0.007	-0.008	2858.5	-2623.9
		6.5	0.024	-0.015	0.020	-0.019		
		11.0	0.048	-0.024	0.043	-0.032		
	CLT-EP	3.0	0.009	-0.005	0.005	-0.007	2358.1	-2219.5
		6.5	0.027	-0.013	0.015	-0.018		
		11.0	0.064	-0.020	0.038	-0.040		
	RIG	3.0	0.009	-0.008	0.007	-0.009	2727.2	-2742.4
		6.5	0.032	-0.019	0.027	-0.024		
		11.0	0.093	-0.034	0.071	-0.058		

The collapse mechanisms can be observed in Fig. 9 and in Fig. 10, where the displacements of the structures are shown for the UR floor and CLT-EL floor cases under the “GMN” earthquake and the “SAN” earthquake. The represented time step is the one when the maximum out-of-plane displacement of the south wall is reached.

In general, analyses of the models with an unreinforced floor clearly show the triggering of an out-of-plane mechanism of the north and south masonry walls. The masonry walls reach out-of-plane displacement values as high as 24 cm (Mirandola 2012 earthquake) or complete collapse (San Felice 2012 earthquake). This can be seen in the out-of-plane displacement curves and as a sudden increase in the kinetic energy.

When the CLT-reinforced floor elastically connected to the masonry walls is considered, the out-of-plane displacements of the masonry walls are usually smaller than those with unreinforced floor. As an example, in Table 3, the out-of-plane displacement of point P1, at the third level of the building, varies from 105 mm (drift 0.76%) in the case of unreinforced floor to 24 mm (drift 0.29%) in the case of the floor reinforced by CLT under the GMN earthquake, from 239 mm (drift 3.73%) to 102 mm (1.74%) under the

MRN earthquake, and there is an out-of-plane collapse under the SAN earthquake, while this is prevented in the model with CLT-reinforced floor. The floor transfers part of the seismic force to the in-plane loaded walls, where shear-sliding failure can be observed. Considering the energy plots, it can be observed that the CLT-reinforced floor is capable of dissipating a considerable amount of energy.

The use of elastic-plastic connections with the same floor type usually means slightly higher out-of-plane displacements and slightly lower base reaction force values. As shown in Fig. 6d, Fig. 7d and Fig. 8d, the energy dissipated by the floor system increases significantly with this connection type. Interestingly, this increase is compensated by a reduction in the energy dissipated by the walls, which means that the masonry is less damaged.

In almost all cases, the use of reinforced floors causes a strong increase in the in-plane displacement of the lateral walls at all levels. This means that more cracks develop in the masonry, which is coherent with the increased transmission of forces to the side walls due to the stiffer floors. The use of elastic-plastic connections is associated with lower in-plane displacements than in the case of elastic floor-to-wall connectors. This confirms the reduced energy dissipation by the walls.

The displacement results of the models with the ideal rigid floor are similar to those observed for the CLT-reinforced floor case. The out-of-plane displacements measured in the rigid floor case are due to the in-plane deformation of the lateral walls and to the deformation of the floor-to-masonry connectors. This comparison shows that the CLT-reinforced configuration is effective in creating a “box behaviour” of the masonry structure. It should be noted, however, that the peak base shear force is often higher when the ideal rigid floor is considered.

In the second series of analyses, non-scaled accelerograms were used, with the goal of testing the structural responses under stronger earthquakes and, in particular, studying the effect of floor-to-wall connections. The two seismic records were previously identified with the names “May76” and “Sep76” from the 1976 Friuli earthquake.

Only the CLT-EL (reinforced floor connected to the walls with elastic connections) and CLT-EP (reinforced floor connected to the walls with elastic-plastic connections) cases have been considered with these two earthquake records.

The results of these DEM analyses are compared in Fig. 11 and Fig. 12. The load transferred from the floors to the lateral walls, the out-of-plane displacements of the face-loaded masonry wall (south and north façade, points P1 and P2 in Fig. 3), the in-plane displacements of the lateral walls (points P3 and P4 in Fig. 3), the base reaction force, and the energy dissipated over time are reported for the two models. The reported energy values are the kinetic energy of the structure, the energy dissipated by the floor hysteretic behaviour and the energy dissipated by the masonry walls due to damage and friction effects.

The results are also summarised in Table 5 and in Table 6, considering the same parameters as before.

Table 5

Summary of the results of the different numerical models (May76 and Sep76 seismic records). Out-of-plane displacements and floor drifts

Earth-quake	Floor type	Height [m]	Out-of-plane displacement P1 [m]		Inter-storey drift [%]		Out-of-plane displacement P2 [m]		Inter-storey drift [%]	
			max	min	max	min	max	min	max	min
May76	CLT-EL	3.0	0.005	-0.003	0.17	0.10	0.003	-0.005	0.10	0.17
		6.5	0.015	-0.013	0.29	0.29	0.014	-0.017	0.31	0.34
		11.0	0.023	-0.050	0.18	0.82	0.037	-0.043	0.51	0.58
	CLT-EP	3.0	0.009	-0.002	0.30	0.07	0.004	-0.007	0.13	0.23
		6.5	0.026	-0.010	0.49	0.23	0.017	-0.019	0.37	0.34
		11.0	0.038	-0.032	0.27	0.49	0.033	-0.035	0.36	0.36
Sep76	CLT-EL	3.0	0.015	-0.028	0.50	0.93	0.013	-0.035	0.43	1.17
		6.5	0.077	-0.102	1.77	2.11	0.057	-0.129	1.26	2.69
		11.0	0.089	-0.462	0.27	8.00	0.086	-0.438	0.64	6.87
	CLT-EP	3.0	0.041	-0.014	1.37	0.47	0.013	-0.044	0.43	1.47
		6.5	0.174	-0.067	3.80	1.51	0.051	-0.190	1.09	4.17
		11.0	0.106	-0.496	-1.51	9.53	0.102	-0.471	1.13	6.24

Table 6

Summary of the results of the different numerical models (May76 and Sep76 seismic records). In-plane displacements and base reactions

Earth-quake	Floor type	Height [m]	In-plane displacement P3 [m]		In-plane displacement P4 [m]		Base reaction [kN]	
			max	min	max	min	max	min
May76	CLT-EL	3.0	0.004	-0.003	0.003	-0.003	1913.0	-2072.2
		6.5	0.010	-0.008	0.008	-0.009		
		11.0	0.015	-0.014	0.008	-0.015		
	CLT-EP	3.0	0.005	-0.003	0.003	-0.003	1875.1	-2221.1
		6.5	0.011	-0.006	0.008	-0.008		
		11.0	0.017	-0.009	0.011	-0.013		
Sep76	CLT-EL	3.0	0.018	-0.028	0.013	-0.029	3613.1	-2871.8
		6.5	0.047	-0.093	0.043	-0.105		
		11.0	0.171	-0.253	0.071	-0.374		
	CLT-EP	3.0	0.016	-0.016	0.010	-0.022	3184.5	-2954.4
		6.5	0.047	-0.062	0.036	-0.118		
		11.0	0.085	-0.216	0.060	-0.375		

The collapse mechanisms can be observed in Fig. 13, where the displacements of the CLT-EL and CLT-EP structures are shown for the Sep76 earthquake. The represented time step is the one when the maximum out-of-plane displacement of the north wall is reached.

The plasticization of the connectors in the CLT-EP case (Fig. 13b) can be observed as a displacement difference between the floor and the top of the lateral walls. These results confirm the possibility of properly designing and calibrating the strengthening intervention to cap the shear forces addressed to the resistant walls and to dissipate energy, simultaneously reducing the out-of-plane displacements of the walls within their capacity.

The following final remarks can be deduced from the reported results:

- DEM allowed to obtain more information about the out-of-plane behaviour of the structure
- Strengthened floors proved to be able to counteract out-of-plane displacements
- The implemented model allows us to estimate the energy dissipated by the floor hysteretic cycles and by the masonry damage

- Strengthened floors are able to dissipate a significant amount of seismic energy, thus reducing masonry damage.

5. Conclusions

Numerical simulations by means of the DEM were carried out to investigate the effectiveness of strengthening solutions for timber floors to enhance the seismic performance of a masonry building. The DEM is an interesting solution to analyse many aspects of masonry structures because of its capability to capture the masonry non-linear behaviour and to follow the collapse mechanisms, with more detailed information than other numerical approaches.

Dynamic non-linear analyses were performed on a case study by considering the floor hysteretic behaviour and different seismic inputs. The building chosen as the case study is a noble villa typical in north-eastern Italy, characterised by the lack of internal walls in one direction.

The analyses were focused on the triggering of 1st mode mechanisms. These are proven to be the governing mechanisms in all the analysed cases with unreinforced floors.

The application of CLT panels, dry connected to the existing floor, was experimentally demonstrated to increase the in-plane stiffness and strength of the floors. The comparison between the numerical results of the unreinforced and reinforced floor cases highlights the effectiveness of the proposed wood-based strengthening solutions in reducing the out-of-plane displacements of the masonry walls to prevent overturning collapse mechanisms. The analyses also highlight the capability of the reinforced floor to transfer the seismic forces to the shear resistant walls by triggering an in-plane shear collapse.

A comparison with the rigid diaphragm case confirms the good performance of the strengthening solution. The observed out-of-plane displacements are compatible with the masonry wall capacity, and energy is dissipated because of the reinforced floor hysteretic cycles.

The effect of different floor-to-wall connections was also assessed. Connections are needed to transfer the load to the bearing walls, but elastic-plastic connections can also be used to cap the load and to dissipate energy. This allows to both reduce the out-of-plane displacement of the face-loaded walls and to limit the in-plane damage to the seismic bearing walls.

In fact, floors that are too stiff could be detrimental to the seismic performance of masonry buildings, and in these cases, it is particularly important to cap the shear forces addressed to the shear resistant walls. By using retrofit solutions such as those considered in this work, the in-plane performance of the floor can be properly designed and calibrated to maximize the energy dissipation without exceeding the capacity of the existing masonry structure.

The combined approach of DEM modelling and timber floor cyclic modelling discussed here has proven to be an interesting strategy to perform further investigations on the possibility of controlling the energy dissipation involved in the dynamic response of masonry buildings with strengthened timber floors.

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Figures

Main façade of the villa (south)

West façade of the villa

Figure 1

Pictures of the Villa Cattaneo (San Quirino, PN, Italy), chosen as a case study

(a) Ground floor and first floor plan

(b) Second floor plan

(c) South façade

(d) East-West cross section

Figure 2

Drawings of the villa chosen as the case study (measures in cm)

Figure 3

Numerical model of the building using the DEM

Figure 4

Simplified modelling approach for the timber floor

(a) Site-compatible spectra

(b) Unscaled seismic records

Figure 5

Response spectra of the three scaled seismic records and the two unscaled records

Figure 6

Numerical results of the different DEM models for the GMN earthquake

Figure 7

Numerical results of the different DEM models for the MRN earthquake

Figure 8

Numerical results of the different DEM models for the SAN earthquake

(a) Unreinforced floor model

CLT-reinforced floor model

(b)

Figure 9

Displacements of the building when the maximum out-of-plane value of the front façade is reached. Scaled seismic record of the Gemona 1976 earthquake (GMN)

(a) Unreinforced floor model

CLT-reinforced floor model

(b)

Figure 10

Displacements of the building when the maximum out-of-plane value of the front façade is reached. Scaled seismic record of the San Felice sul Panaro 2012 earthquake (SAN)

Figure 11

Numerical results of the different DEM models for the May76 earthquake

(a) total force transmitted to the lateral walls – elastic case

(a) total force transmitted to the lateral walls – elastoplastic case

(a) out-of-plane displacement P1.3 – comparison

(b) in-plane displacement point P4.3 – comparison

(c) base reaction - comparison

(d) energy - comparison

Figure 12

Numerical results of the different DEM models for the Sep76 earthquake

Figure 13

Displacements of the building when the maximum out-of-plane value of the rear façade is reached.
Gemona 1976 earthquake – September 15th