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A seismic retrofitting design approach for activating dissipative behavior of timber diaphragms in existing unreinforced masonry buildings

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ABSTRACT: The region of Groningen (NL) has experienced increasing human-induced seismicity caused by gas extraction in the last decades. The local building stock, not designed for seismic loads, consists for more than 50% of unreinforced masonry buildings with timber diaphragms. In this context, a detailed seismic characterization of timber and masonry structural components has taken place, and a retrofitting technique for timber floors activating their energy dissipation has been developed. Besides, specific analytical and numerical modeling strategies for as-built and retrofitted timber floors have been formulated. This work presents a design approach for creating strengthened dissipative timber diaphragms, and maximizing the seismic capacity of existing masonry buildings through this retrofitting method. The results from the performed numerical analyses prove that the proposed design approach for timber floors can increase the energy dissipation capacity of masonry buildings, while improving the box behavior at both damage and near-collapse limit state.

1 INTRODUCTION

In the northern part of the Netherlands, within the Province of Groningen, human-induced earthquakes caused by gas extraction have occurred in the last decades (van Eck et al. 2006). Because these events were absent until recently, the local building stock was not designed or realized accounting for seismic loads. These buildings consist for more than 50% of unreinforced masonry (URM) structure with slender walls and poorly connected, flexible timber floors.

Thus, the existing structural components needed to be seismically characterized, and proper retrofitting measures to be developed. In this context, a detailed assessment of masonry (Messali et al. 2017, Jafari et al. 2017) and timber (Mirra et al. 2020) structural components has taken place at Delft University of Technology since 2016. With specific focus on timber diaphragms, in-plane tests on floors loaded parallel and perpendicular to the joists, as well as on a roof sample, were performed (Mirra et al. 2020). As-built floors showed a very flexible behavior, and in order to increase their in-plane stiffness and shear transfer capacity, a retrofitting technique was developed, consisting of plywood panels screwed along their perimeter to the existing sheathing. This strengthening solution has been designed on the basis of promising results obtained in similar studies from other seismic contexts (Peralta et al. 2004, Brignola et al. 2012, Giongo et al. 2013, Wilson et al. 2014), with the aim of increasing not only strength and stiffness of the diaphragms, but also their energy dissipation (Fig. 1).

This dissipative behavior could be achieved for the strengthened diaphragms, and both analytical (Mirra et al. 2021a, b) and numerical (Mirra et al. 2021c, Mirra & Ravenshorst 2021) modeling strategies were developed for an advanced simulation of their nonlinear in-plane response (Fig. 1). These formulations open up the opportunity of designing the diaphragms in such a way that they can fully transfer the expected seismic shear forces, while activating a beneficial energy dissipation. In other words, the plywood panels retrofitting is not adopted to create rigid diaphragms in existing URM buildings, but dissipative structural components, able to decrease seismic actions on the walls.

Thus, this work presents a design approach for retrofitting timber floors, aimed at maximizing the seismic capacity of existing URM buildings through the activation of hysteretic energy dissipation in the diaphragms. The adopted methodology for designing and modeling the floors is described in Section 2 and exemplified in Section 3, with specific focus on the effect of the size of plywood panels and the number or diameter of screws on the in-plane response of the diaphragms. In Section 4, the design approach is validated with time-history analyses on three case-study URM buildings, evaluating the results at near-collapse and damage limit state. Finally, Section 5 presents the conclusions of this study and recommendations for future research.

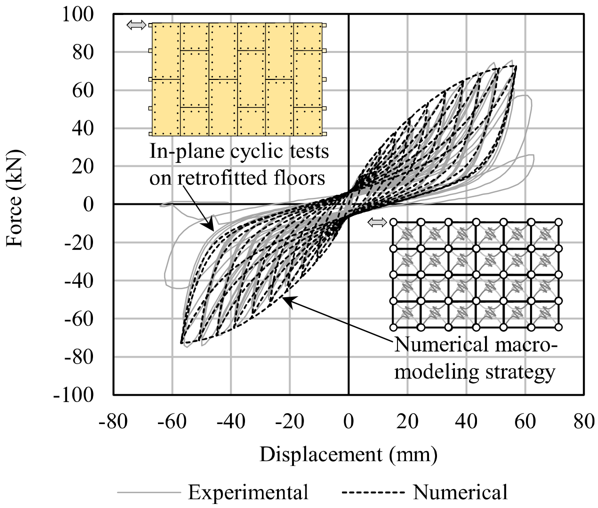

Figure 1. In-plane cyclic test on a timber diaphragm retrofitted with plywood panels and its numerical modeling. The large energy dissipation activated by the fasteners is evident.

2 DESIGN METHODOLOGY

The design approach presented in this section refers to URM buildings sufficiently regular in plan and elevation according to EN 1998, and is schematically presented in Figure 2. Once that a preliminary (on-site) investigation on geometrical and material properties of the building is conducted, the first step consists of estimating its seismic capacity in terms of base shear. To this end, a nonlinear static (pushover) analysis can be conducted, preliminarily considering rigid diaphragms (Fig. 2a).

After determining the base shear F_b in the weakest direction, the corresponding seismic forces F_i at each floor level are calculated with the lateral force method (Fig. 2b):

$$F_i = F_b\, h_i\, W_i / (\Sigma_i\, h_i\, W_i) \tag{1}$$

with h_i and W_i height and seismic mass of the floor level i, respectively. The shear forces F_i constitute the design seismic loads to be transferred by each retrofitted diaphragm without causing an out-of-plane collapse of masonry walls. In other words, these forces represent the minimum value of the strength of the diaphragms, according to which a sufficient number of fasteners for applying the plywood panels overlay can be designed.

In order for the energy dissipation to be activated, the diaphragms should display in-plane deflection capacity, but without causing the out-of-plane collapse of masonry walls. Therefore, besides the definition of the strength of the retrofitted floors following Equation 1, also a displacement limit to avoid an excessive deflection is set (Fig. 2c). This value could be arbitrarily chosen depending on the conditions and geometry of the out-of-plane masonry walls. With reference to NZS 1170 (2004), in principle the limit should not overcome half of the thickness of the walls, and the total horizontal deflection of the building along the earthquake direction should not be higher than 2.5% of the height of the construction (NZS 1170 2004). This same limit has then been implemented in Dutch seismic guidelines NPR 9998 (2020). On the basis of these proposed limits, in the present work the following values have been considered for determining the maximum in-plane deflection $\delta_{max,i}$ of each retrofitted diaphragms:

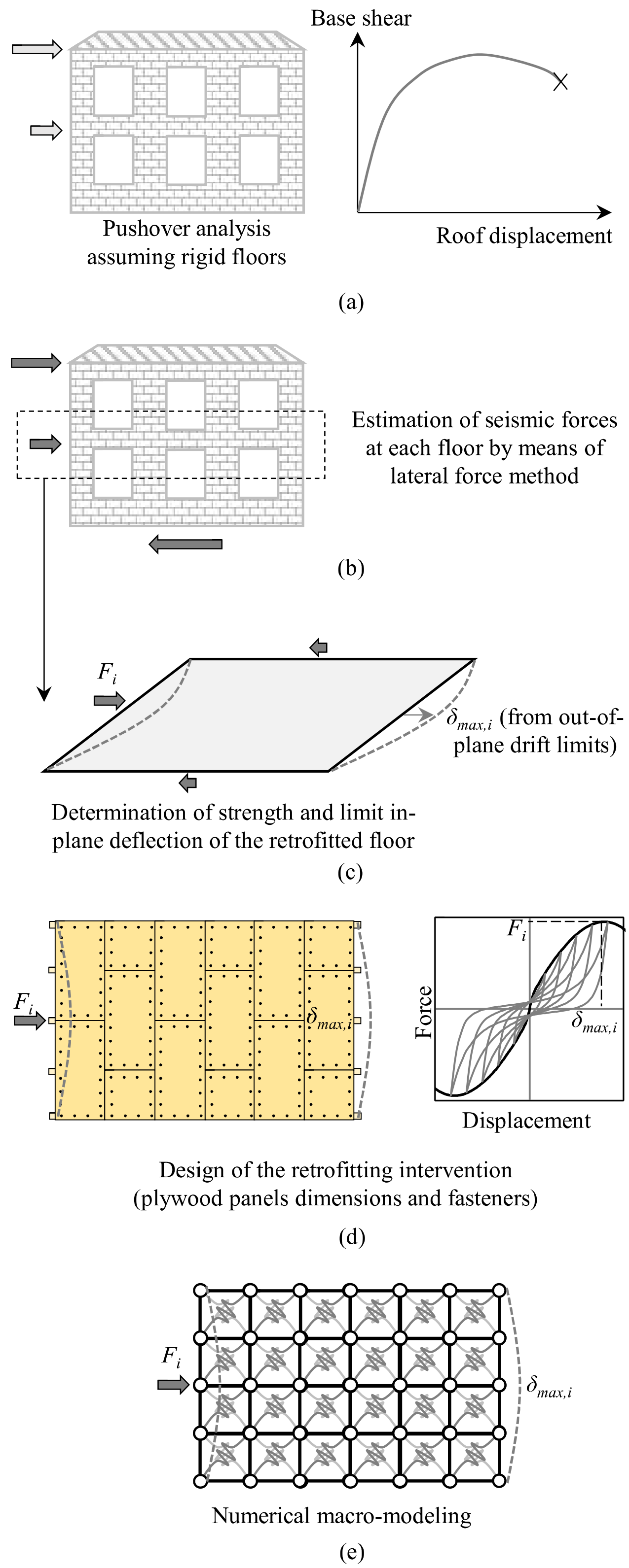

Figure 2. Design approach for retrofitting and modeling diaphragms activating energy dissipation in URM buildings.

$$\delta_{max,i} = \min(t_w/2;\ 0.02h_w) \quad (2)$$

where t_w and h_w are the thickness and absolute height of the out-of-plane masonry walls. This means that, as an example, in presence of a single-leaf slender wall with t_w = 100 mm and h_w = 3000 mm, δ_{max} = $t_w/2$ = 50 mm, whereas for thicker walls the drift limit related to h_w becomes governing.

Once that the in-plane strength and limit deflection are known for the diaphragm, the retrofitting intervention is designed by considering the floor as a timber shear wall: the analytical procedure presented in Mirra et al. (2021a) can be followed. In this way, starting from the load-slip response of the single fastener connecting plywood panels and planks, by means of equilibrium relations the backbone curve and pinching cycles of the whole diaphragm can be determined (Fig. 2d). It should be noticed that the in-plane strength and stiffness of the retrofitted floor is governed by the number and diameter of fasteners, but also by the dimensions of the plywood panels. Therefore, in agreement with the specific design choices, it is possible to create stiffer or more flexible floors, and to predict their energy dissipation and pinching cycles accordingly (Fig. 3). More specifically, with respect to a reference designed retrofitted configuration of a 4.0×4.6 m floor (Fig. 3b), the strength is mostly governed by the number or diameter of screws (Figs 3c, e), whereas the displacement capacity is influenced by the width of the panels (Fig. 3d). In principle, taking into account these effects, the design choice maximizing the hysteretic energy that can be activated, produces a floor that reaches its target in-plane strength F_i (corresponding to the expected seismic shear load) at the displacement limit $\delta_{max,i}$ preventing out-of-plane collapse of masonry (see again Fig. 2d).

In numerical analyses, the designed retrofitted floors can then be conveniently modeled following a macro-element approach (Mirra et al. 2021c). A floor is subdivided in a mesh of quadrilaterals composed of rigid truss elements, with two diagonal truss elements featuring the constitutive law simulating the in-plane response of the diaphragm (Fig. 2e). Therefore, these nonlinear diagonal elements are assigned a load-slip response determined on the basis of that analytically calculated while designing the floor, and derived by means of geometrical relations. Since the macro-elements only simulate the in-plane behavior of the floors, the out-of-plane response (under vertical loads) is modeled with linear elastic orthotropic plate elements, on which the macro-elements are overlapped (Mirra et al. 2021c, Mirra & Ravenshorst 2021).

The next section presents an example of application of the aforementioned procedure for a reference floor to be retrofitted. Subsequently, the influence of dissipative diaphragms in URM buildings is discussed in Section 4.

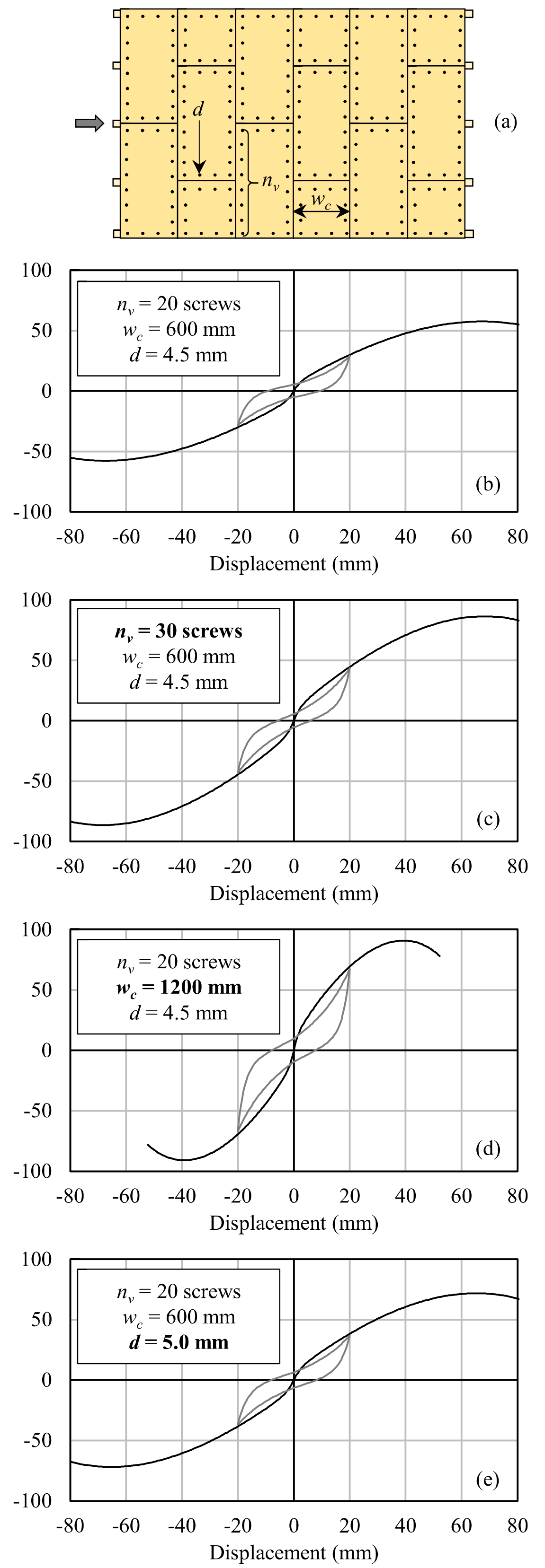

Figure 3. Influence of number n_v and diameter d of screws and plywood panels width w_c on the in-plane response of the floors: (a) reference parameters; (b) reference backbone curve and pinching cycle for a 4.0×4.6 m retrofitted floor; (c) effect of number of screws; (d) effect of plywood panels width; (e) effect of diameter of screws.

3 DESIGN EXAMPLE

To better illustrate the design process of the retrofitting technique activating energy dissipation in timber floors, the methodology described in the previous section is now applied in a reference example.

The optimal retrofitting intervention should combine a sufficient strength with this displacement capacity, so that the maximum energy dissipation can be retrieved: in this way, it is possible to create dissipative diaphragms activating an equivalent hysteretic damping ratio of 15% (Mirra et al. 2021d). In order to prevent out-of-plane collapses in the masonry, besides limiting the deflection of the diaphragms, it is also necessary to create continuous floor-to-wall joints, for which several strengthening options are available (Mirra et al. 2021e).

Consider a $B \times L$ = 4.0×4.6 m floor with 18-mm-thick planks to be retrofitted, and subjected to a design seismic shear of 100 kN. The inter-story height and thickness of the masonry walls are 3.0 m and 210 mm, respectively.

Therefore, the retrofitting system has to withstand and transfer 50 kN for each supported side, and its maximum in-plane displacement to prevent the walls from out-of-plane collapse is 60 mm, following the limits discussed in Section 2. With reference to Figure 4 and adopting the same notation as in Mirra et al. (2021a), assuming a 100 mm spacing for the screws, the number n_v of fasteners in half the floor span is 23, placed at a distance e = 50 mm from the side of the panel.

Considering symmetry, the expected in-plane shear force in the floor causes a reaction of 50 kN at both supported sides. This force is further subdivided among the n_c panels' columns. At the same time, the displacement δ_{max} = 60 mm at which the in-plane strength should be reached, induces a rotation (Fig. 4) $\vartheta = \delta_{max}/(L/2) = d_{s,v}/(w_c - e)$, with $d_{s,v}$ slip of the screws at their strength (Mirra et al. 2021a).

Assuming 4.5×40 mm screws, an overlay of 18-mm-thick structural plywood panels, and an average density of 450 kg/m^3 for timber, following EN 1995 and the analytical formulation in Mirra et al. (2021a), a single screw results in a strength $F_{s,v}$ = 1.6 kN and $d_{s,v}$ = 13.7 mm.

Therefore, n_c = 6 columns of panels ($w_c \approx$ 670 mm) are necessary to fulfil the imposed in-plane displacement limit. Furthermore, the total strength of half of a floor is sufficient to withstand the expected seismic load: $n_v F_{s,v}(w_c - e)n_c/(L/2)$ = 56.9 kN, which is higher than the expected reaction force of 50 kN.

The retrofitted configuration resulting from the design is represented in terms of backbone curve and internal pinching cycles in Figure 5, constructed according to the procedure elaborated in Mirra et al. (2021a).

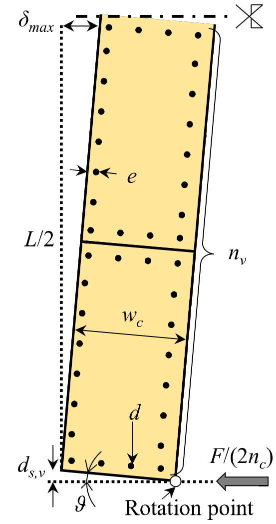

Figure 4. Scheme showing a column of plywood panels fastened to the existing floor, and adopted for the design of dissipative retrofitted diaphragms.

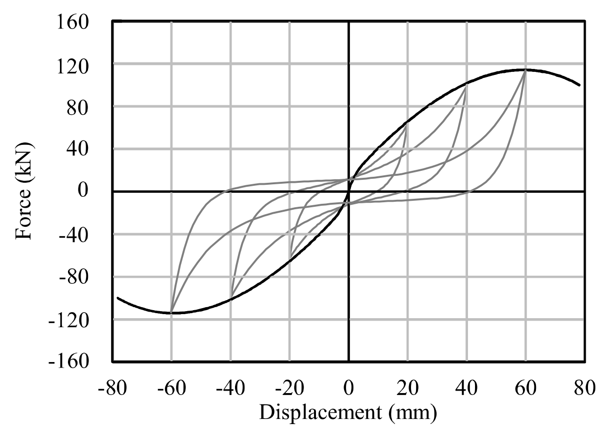

Figure 5. Backbone curve (black) and internal pinching cycles (grey) of the designed retrofitted diaphragm.

4 APPLICATION OF THE DISSIPATIVE FLOORS IN URM BUILDINGS

The design principle presented in the previous section was applied in three case-study buildings (Fig. 6), two having Dutch features and one belonging to the Italian context, in order to further generalize the proposed procedure. For additional details regarding the choice and modelling strategies of the buildings, the reader is referred to Mirra et al. (2021c), Mirra & Ravenshorst (2021); in this section, the results and impact of floors retrofitted to retrieve the maximum energy dissipation in URM buildings, are discussed.

The case-study buildings were analyzed considering three configurations: an as-built one, with existing flexible floors; one featuring rigid diaphragms realized with the casting of a concrete slab on the existing floors; one in which the retrofitting solution activating energy dissipation was applied. In order to realistically capture the seismic response of the case-study buildings, each configuration was subjected to seven seismic accelerograms applied in both plan directions.

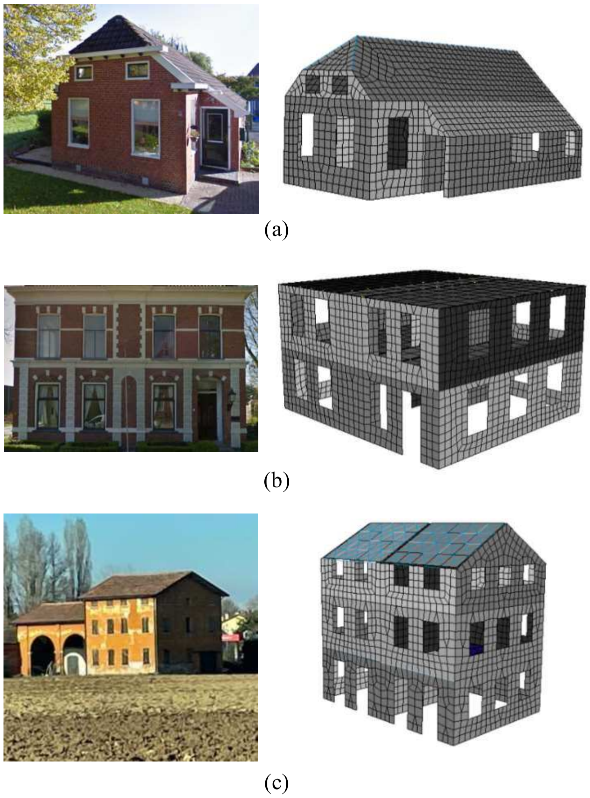

Figure 6. Case-study URM buildings and their numerical models adopted for analyzing the impact of dissipative diaphragms on their seismic response: building B1 (a), building B2 (b), building B3 (c).

All numerical time-history analyses were conducted in the finite element software DIANA FEA v. 10.4 (Ferreira 2020). Table 1 reports the properties adopted for masonry, whereas Table 2 presents the reference values of seismic shear and maximum in-plane deflection according to which the dissipative diaphragms were designed. All walls were simulated with shell elements featuring the Engineering Masonry Model (Ferreira 2020), while the floors were modeled with linear elastic orthotropic plates, with the exception of the dissipative diaphragms, for which macro-elements were used (see for further details Mirra et al. 2021c, Mirra & Ravenshorst 2021).

Table 1. Properties of masonry adopted in the numerical models of the three case-study buildings.

Property	B1	B2	B3
Thickness (mm)	210	210	380
Young modulus parallel to the bed joint (MPa)	1500	1500	1875
Young modulus perpendicular to the bed joint (MPa)	2000	2000	2500
Shear modulus (MPa)	800	800	1000
Mass density (kg/m^3)	2000	2000	2000
Bed joint tensile strength (MPa)	0.15	0.15	0.15
Fracture energy in tension (N/mm)	0.01	0.01	0.01
Compressive strength (MPa)	14.0	14.0	8.0
Compressive fracture energy (N/mm)	30	30	35
Friction angle (°)	34	34	34
Cohesion (MPa)	0.2	0.2	0.2
Fracture energy in shear (N/mm)	0.1	0.1	0.1

Table 2. Values of seismic shear loads and in-plane displacement limits for designing the dissipative diaphragms in the three analyzed buildings.

Property	First floor	Second floor	Roof
Building B1			
Strength (kN)	200	N.A.	150
Displacement (mm)	60	N.A.	40
Building B2			
Strength (kN)	330	N.A.	330
Displacement (mm)	70	N.A.	140
Building B3			
Strength (kN)	230	450	330
Displacement (mm)	60	120	180

The results in terms of achievable peak ground acceleration (PGA) at collapse show that the impact of dissipative diaphragms is very beneficial for the case-study buildings, because of the additional energy dissipation and damping effect on masonry walls: on average, the highest values of PGA at collapse are retrieved for these retrofitted cases (Fig. 7a). It should be noticed that a major improvement in the seismic response and box behavior of the analyzed buildings is already retrieved with rigid concrete floors. However, the additional dissipation of the designed plywood panels solution results in an increase of the PGA at collapse of 30% on average (Fig. 7b).

This outcome confirms the obtained 15% equivalent damping ratio value for these floors (Mirra et al. 2021d): if the floors were retrofitted with plywood panels, and for a simplified modeling (e.g., a pushover analysis), they were assumed as stiff; their dissipative contribution could be taken into ac-count by considering an overdamped spectrum reduced by the factor $\eta = [10/(5 + \xi)]^{1/2}$ (EN 1998). It is interesting to notice that $\eta = 0.707$ for $\xi = 15\%$, a value suggesting that, when a dissipative retrofitting of the floors is designed, the demand response spectrum could indeed be reduced by approximately 30% in addition to the further nonlinear contributions of the in-plane masonry walls. The additional dissipation induced by the floors designed following the proposed approach is a relevant part of the hysteretic energy activated in the whole building (Fig. 8). This confirms that dissipative diaphragms, able to retrieve the box behavior in the building, can be a better alternative than rigid diaphragms, concentrating the dissipation only in masonry walls.

Besides, although the retrofitted dissipative floors are deformable in plane, in the analyses the out-of-plane drift of masonry at damage limit state did not overcome the limit of 0.33% of the wall height (D'Ayala 2013), with only minor cracks caused to the wall, similarly to those occurring with concrete slabs. Instead, with existing floors these requirements could not be met. The presented design method can thus be suitably adopted for a retrofitting balancing strength, stiffness, and energy dissipation of floors at both near-collapse and damage limit state.

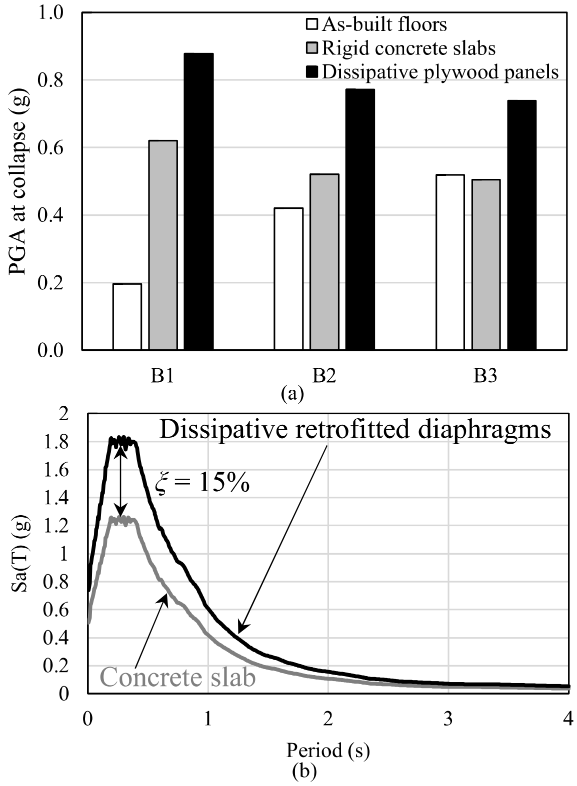

Figure 7. Average PGA at collapse of the studied configurations (a); response spectra referred to building B3, highlighting the dissipation of the floors compared to rigid diaphragms (b).

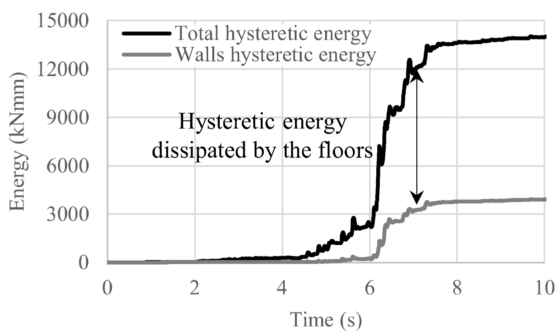

Figure 8. Example of hysteretic energy activated by the dissipative retrofitted diaphragms, with reference to building B1.

5 CONCLUSIONS AND OUTLOOK

In this work, a design approach for retrofitting timber diaphragms in existing URM buildings has been shown. This method allows to activate additional energy dissipation in the floors, which can greatly improve the seismic performance of the buildings. Besides showing a design example, the results of numerical time-history analyses on three case-study URM buildings have been discussed. It has been proved that diaphragms retrofitted with the proposed design approach are indeed able to activate large hysteretic energy dissipation, quantified with an equivalent damping ratio of 15%. For future research, it is recommended to further validate these results, and to evaluate the effect of such retrofitted diaphragms in terms of behavior factor, also by means of experimental tests. The obtained results can contribute to the research framework supporting the conservation of the architectural heritage of seismic-prone countries.

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