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DYNAMIC SIMULATION OF AN IRREGULAR MASONRY BUILDING WITH DIFFERENT REHABILITATION METHODS APPLIED TO TIMBER FLOORS

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Abstract. *The in-plane stiffening of timber floors is normally supposed to be an improvement of the seismic performance of un-reinforced masonry buildings. A modelling strategy to simulate the non-linear behaviour of masonry buildings with simple or strengthened timber floors is presented: it allows to implement the in-plane hysteretic response of the floors and different types of failure of the masonry walls. This model was used to predict the modification of the seismic response of a two-storey masonry building subjected to different rehabilitation techniques applied to the timber floors. The case-study building is irregular in plan to study also torsional effects and out-of-plane deformation of the walls. The mechanical parameters of the non-linear elements representing masonry piers and floors were calibrated replicating experimental tests available in literature. The outcomes of this work were obtained with non-linear dynamic analyses, in order to allow the model to consider not only the actual elastic and post-elastic stiffness of the floors but also their energy dissipation capacity.*

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1 INTRODUCTION

Various techniques are available and are commonly used to improve the seismic response of un-reinforced masonry (URM) buildings. Among them, the stiffening and strengthening of timber floors in their plane are normally supposed to be an improvement of the global seismic response of the building. The most common methodologies are available in ([1]-[3]). Various monotonic or cyclic-loading tests have been performed to evaluate the in-plane behaviour of strengthened timber floors ([1], [4]-[5]) and the results can be used to perform numerical models of entire buildings, without neglecting the actual stiffness and the hysteretic behaviour of the floors.

Numerical studies, which simulated the seismic response of masonry buildings with strengthened timber floors, are available in literature (e.g., [6], [7]). Results obtained in these works show that the in-plane stiffening of timber floors improves the behaviour of the building and its global strength, if the failure of the not-strengthened building was due to out-of-plane overturning of walls. These numerical studies were performed via non-linear static analyses; therefore, they do not consider the hysteretic behaviour and the dissipative capacity of the floors. Nevertheless, the hypothesis that the stiffening of timber floors can be in some cases unfavourable and could worsen the seismic performances of the building has been already stated (e.g., [1], [6]).

Numerical studies were recently performed by means of non-linear time-history analyses with 3D models ([8]-[9]). In both works, only the walls parallel to the imposed earthquake direction were modelled. In [8] three case-study buildings with slender masonry piers have been studied and effects of spandrels have been neglected, in order to reach always a rocking failure. The buildings differed in the plan configuration of the masonry piers. In [9] another building has been modelled. In this case, the actual non-linear behaviour of all possible failure mechanisms of the masonry was considered (i.e., rocking, sliding and diagonal cracking) and effects of spandrels were not neglected. These two research works analysed different buildings with a different finite-element software but the same conclusion was obtained: if the out-of-plane failure of masonry walls is avoided, the seismic capacity of a URM building can decrease if a retrofitting method leading to excessive floor stiffening is adopted. The use of time-history analyses allowed the model to consider the dissipative capacity provided by the floors and therefore the reduction of forces transmitted to shear walls, and it was observed that the building is not able to exploit such dissipative capacity, if the in-plane stiffness of the floors is too much increased.

In this work, another two-storey building has been analysed with time-history simulations. The model takes into account the actual hysteretic behaviour of the floors, the in-plane and out-of-plane beahviour of all the façades (parallel and orthogonal to the earthquake direction), and all possible failure modes of masonry.

2 DESCRIPTION OF THE NUMERICAL MODEL

2.1 Geometry and model of the building

The case-study building has the same geometrical configuration of the 2:3 scaled building analysed in [6] but real dimensions. It has a rectangular plan with dimensions 4.00 x 4.80m and two-storeys, with inter-storey height equal to 3.00m. The structural thickness of the walls equal to 38.0cm and their mechanical properties were chosen to be consistent with the experimental tests used to calibrate the non-linear behaviour of masonry (see section 2.2). The building is shown in Figures 1 and 2.

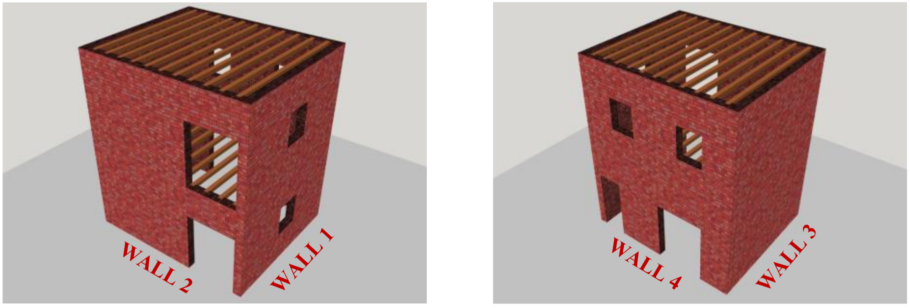


Figure 1: 3D views of the building's structure.

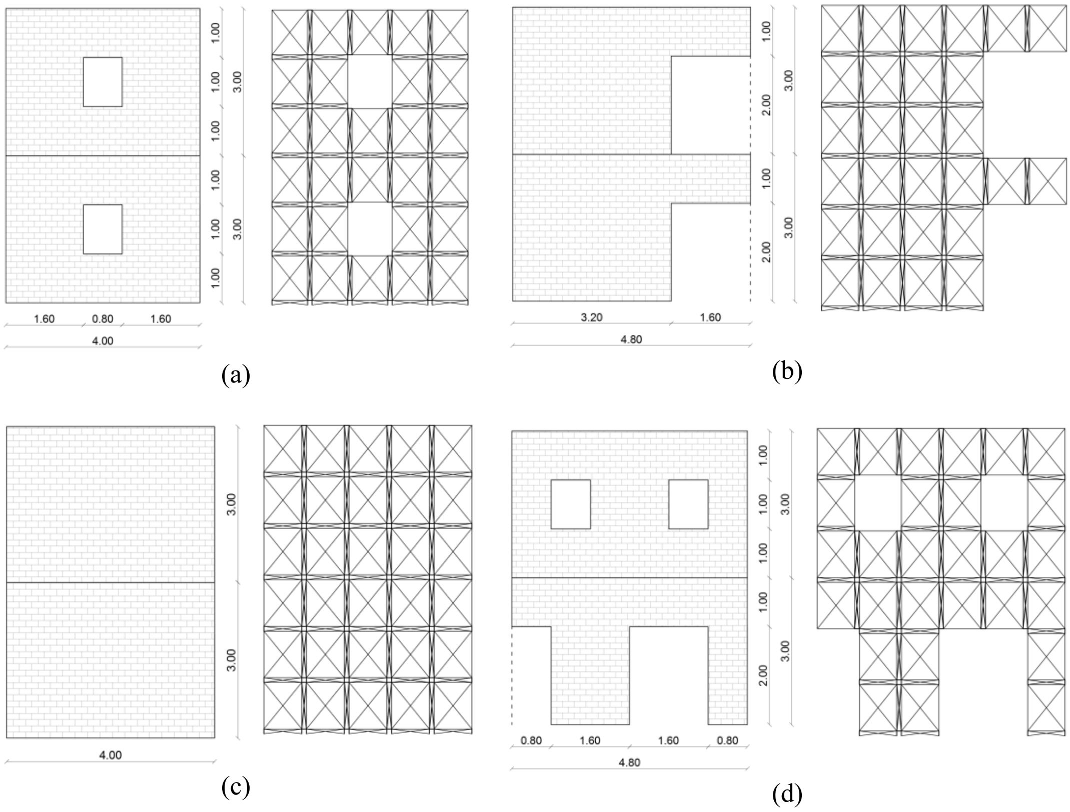


Figure 2: Façades and model of the building. (a) Wall 1; (b) Wall 2; (c) Wall 3; (d) Wall 4.

The original floors of the building are supposed to be composed of a single layer of 20x3cm C22-class timber boards nailed orthogonally to 18x18cm GL24c-class timber beams having spacing of 50 cm with four nails per intersection.

The effects of three stiffening techniques in comparison with the original floor have been analysed. The first technique consists of an additional layer of 30mm thick timber boards arranged at 45° with respect to the original boards and fixed to the beams with 6x90mm timber screws (from 2 to 4 screws per intersection). The second technique consists of the addition to the original floors of light-gauge steel plates (80x2mm) screwed to the existing boards at 45° with 5x25mm screws (20 screws per meter). The mesh of diagonal bracing is 705mm. The last analysed technique is the addition of a 50-mm thick RC slab reinforced with 6-mm diameter rebars (mesh 200x200mm). The connection between timber beams and RC slab is assured by 14-mm diameter L-shaped steel bars spaced 20-30cm glued with epoxy resin. The choice of such details of the retrofitting strategies has been made to be consistent with the experimental cyclic-loading tests used to calibrate the non-linear behaviour of floors (see section 2.3). The numerical FE model of the building was implemented into the research-oriented numerical code “Open System for Earthquake Engineering Simulation - OpenSees” [10] assembling linear and non-linear truss elements, to form a lattice structure. Two macro-models have been used to simulate floor and masonry behaviour respectively, which are reciprocally connected with rigid hinges.

2.2 Masonry model and calibration

All the URM walls of the buildings were modelled assembling the main modules composed of a hinged quadrilateral perimeter with rigid-elastic behaviour and non-linear diagonal springs accounting for shear behaviour and possible diagonal shear cracking. The masonry modules are reciprocally connected by means of non-linear elements to simulate possible sliding due to shear stress with low tensile strength. The modules are also connected to the foundation by means of non-linear uniaxial springs accounting for possible rocking or sliding failures. Each module representing masonry piers has dimensions 0.80x1.00m (Figure 3a).

Hysteretic material laws were used to have the possibility of representing hardening and softening branches, zero-tensile strength and failure conditions. The non-linear elements have the following properties:

- Symmetric non-linear elements with post-elastic softening branch and imposed failure strain to simulate the possible diagonal shear failure. The constitutive law takes into account the effects of vertical loads acting on the main module;
- Compression-only elements with elastic perfectly-plastic behaviour and ultimate failure strain to simulate the possible failure due to compressive stress at the corner of the masonry modules;
- Non-linear elements with symmetric elastic perfectly plastic behaviour with imposed failure strain to simulate failure due to sliding. The constitutive law takes into account the effects of vertical loads acting on the main module.

It is worth noting that the modules representing spandrels have a slightly different behaviour than the modules representing masonry piers, due to the absence of vertical compression. Moreover, to allow spandrels to work properly in their plane, the presence of passive perimeter steel ties at each storey was assumed and modelled with truss elements characterized by the actual tensile stiffness of the ties.

The mechanical parameters of the non-linear springs representing the masonry piers were calibrated by replicating experimental tests of specimens showing different failure modes [11]. Fig. 3b shows the correspondence between calibration results and experimental data for a specimen characterized by diagonal shear failure.

The out-of-plane stiffness of the walls was simulated with a mesh of horizontal and vertical beams superimposed on the masonry modules (Fig. 3c). These beams have negligible stiffness in their axial direction and in the direction parallel to the façade; therefore, they do not modify the in-plane behaviour given by the main modules. To simplify the model, the beams were modelled with linear elastic behaviour; therefore, they do not simulate failure or damage phenomena. A limit of 2.0% inter-storey drift was assumed as failure due to out-of-plane overturning.

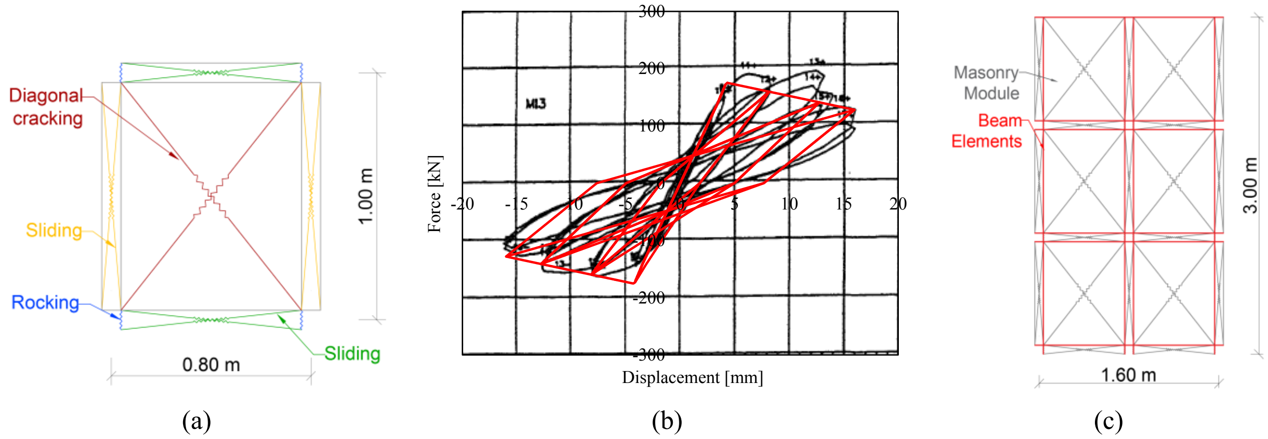


Figure 3: (a) Masonry module; (b) Numerical validation of masonry model: numerical results (in red colour) superimposed on test data from [11]; (c) Out-of-plane beams.

2.3 Floor models and calibrations

The same modelling strategy adopted for masonry was used to simulate the in-plane shear deformability of the floors. In this case, a phenomenological model with the non-linear hysteretic behaviour concentrated exclusively in the equivalent diagonal elements was adopted. This modelling strategy has already been used in literature to simulate the non-linear behaviour of deformable timber shear walls (e.g., [12]). The floor models were obtained subdivided the plan dimensions into a mesh of 6x4 floor modules (Figure 4a), having dimensions of 0.8x0.8m, in order to distribute the nodal masses (Figure 4b) and to fit the discretization used in the wall modelling. Pinching4 model [13] was used to simulate the actual in-plane hysteretic behaviour of the floors characterized by the pinching phenomenon typical of dowel-type fasteners embedded in timber members and cyclically loaded in shear. The non-linear parameters of the floor modules were calibrated fitting the forces-displacement curves obtained from results of cyclic-loading tests available in literature ([1], [14]), Figure 5.

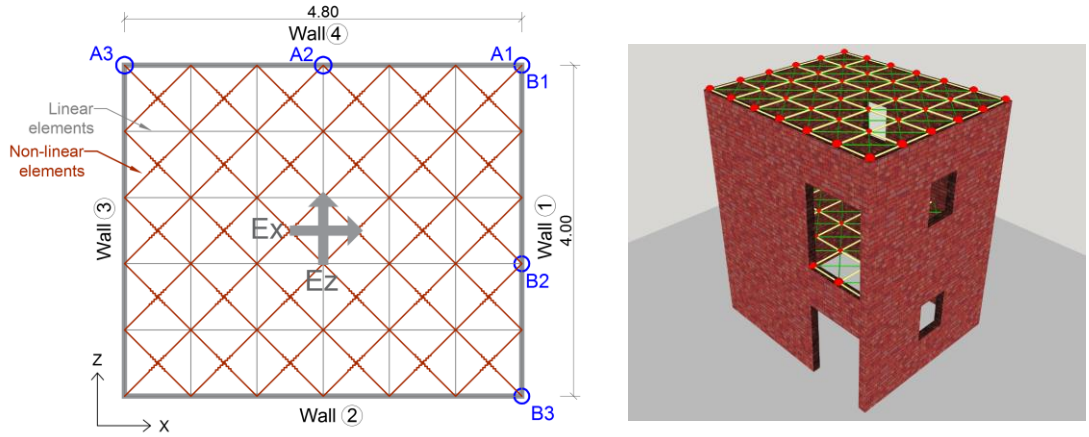
Mass and vertical loads have been computed according to the seismic combination of EN 1990 [15] assuming live loads equal to 2.00kN/m^2. Translational point masses have been arranged in the floor nodes according to the relative afferent areas (Figure 4b). Wall mass has been arranged at the perimeter of the floors.

Three hypotheses were necessarily assumed, which confer specific characteristics to the case-study building:

- The in-plane orthotropic behaviour of the floors has been neglected because only data from tests of floors loaded parallel to beam direction were available. Therefore, the floor beams of the case-study building are supposed to be always parallel to the earthquake direction;
- The in-plane shear stiffness of floors considers only the effects of the structure, whereas possible stiffening effects due to the presence of a lightweight screed or finishing materials were neglected. This implies that in the case-study building only screeds with loose materials are allowed;
- The elastic truss elements at the perimeter of the floor modules with an imposed high axial stiffness, necessary to allow the floor to deform correctly in shear, connect the parallel

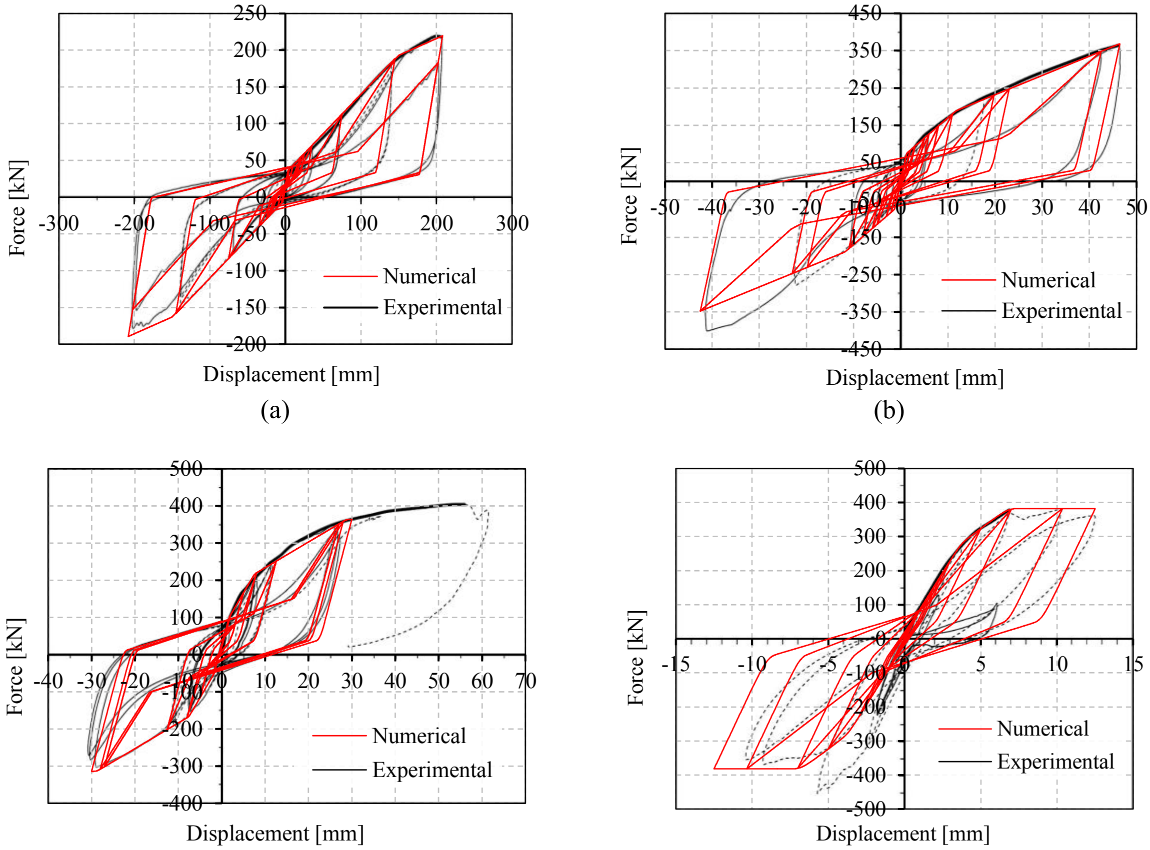
walls reciprocally (see grey lines in Figure 4a). This implies a better seismic response of the case-study building and an improved out-of-plane response of the walls.

These hypotheses, together with the presence of the passive steel ties at spandrel levels (see section 2.2) and the rigid connection between floors and walls (see section 2.1) are fundamental to read correctly the results of this work.



(a) (b)

Figure 4: (a) Floor modules and measurement points for the analyses; (b) distribution of translational masses.



(a) (b)

(c) (d)

Figure 5: Calibration of the in-plane behaviour of the original not-strengthened floor (a) and of the strengthened floors (b)-(d). In detail: (b) addition of 45° timber boards; (c) addition of steel plates; (d) addition of an RC slab.

3 NUMERICAL RESULTS

Non-linear analyses were performed to compare the effects of the rehabilitation techniques among them and with the original floor. For each configuration, non-linear static analyses (NLSA) and incremental dynamic analyses (IDA) with increasing PGA level were conducted. The failure condition has been defined as the first failure of a non-linear element, i.e., the achievement of its ultimate displacement, or the exceedance of the imposed limit of the out-of-plane deformation.

3.1 Non-linear static analyses

NLSA were performed for each configuration and for both the directions E_x and E_z according to Figure 4a. An increasing displacement was imposed to both the floors of the building with a triangular distribution. At the end of the analyses, the failure of the building was reached always due to diagonal shear failure of wall 1 (E_z direction) or wall 4 (E_x direction) at the first storey, whereas out-of-plane limit was never reached. The control points were placed at the middle span of the floors and above the walls parallel to the imposed direction, according to Figure 4a (points A1 to A3 for E_z and B1 to B3 for E_x). They were controlled both at the top of the building (hereafter identified with the subscript "_top") and at first floor level (hereafter identified with the subscript "_mid"). Therefore, six control points were monitored during the analyses for each direction. The six capacity curves are reported in Figures 6 and 7 for each type of floor and for both directions. The need of plotting multiple curves is due to the in-plan irregularity of the building and the presence of in-plane deformable floors. These two conditions limit the validity of the pushover analysis and make necessary a detailed time-history analysis.

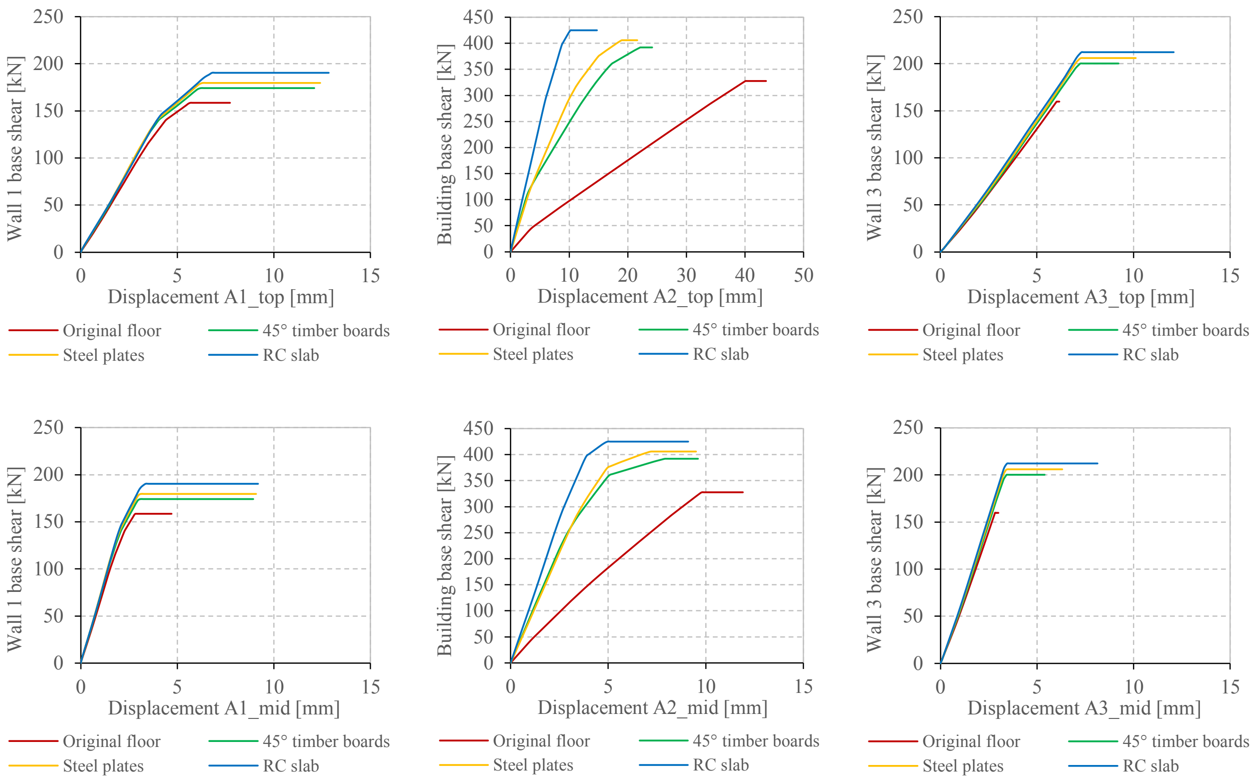


Figure 6: Results from pushover analyses in E_z direction.

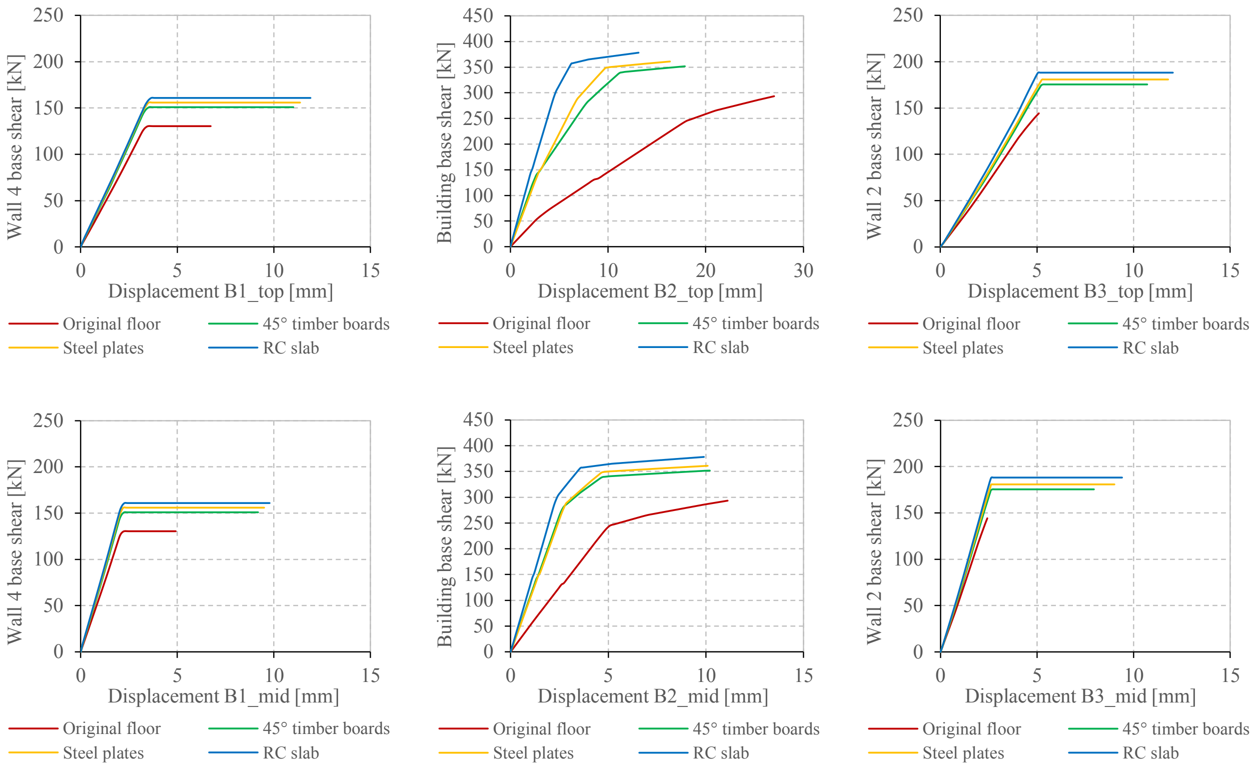


Figure 7: Results from pushover analyses in E_x direction.

A first conclusion from pushover analyses is that increasing the in-plan stiffness of the floors, the global stiffness and strength of the building and of shear walls increase whereas the relative displacements of the floors strongly decrease. It can be noted also that increasing the in-plan stiffness of the floors, the displacements of the weakest shear wall tend to match the displacements of the stronger shear wall, due to a better distribution of forces; this implies a better behaviour of the strengthened buildings in terms of strength and displacement capacity of the walls.

The different seismic performance of the analysed configurations was quantified computing the ultimate PGA compatible with a spectrum for building foundations resting on type A soil and maximum spectral amplification factor F_0 equal to 2.5. The capacity curves of the weakest walls were analysed, i.e., wall 1 base shear *vs.* displacement A1_top for E_z direction and wall 4 base shear *vs.* displacement B1_top for E_x direction. The PGA values are listed in Table 1. These results seem to demonstrate that the seismic performance of this building has been improved increasing the stiffness of the floors. However, these results do not consider the dissipative capacity of the floors.

Floor type	E_z direction	E_x direction
Original	0.47g	0.40g
45° timber boards	0.62g	0.55g
Steel plates	0.64g	0.57g
RC slab	0.67g	0.59g

Table 1: Results from pushover analyses in terms of ultimate PGA.

3.2 Non-linear dynamic analyses

The IDA were performed for all configurations considering an artificial earthquake [16] generated respecting the spectrum compatibility requirements according to the elastic response spectrum 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 incremented with small-amplitude steps from about 0.05g up to the near-collapse condition, corresponding to the ultimate displacement of a non-linear element. The analyses were carried out only in E_x direction (i.e., parallel to walls 2 and 4), which resulted to be the weak direction from pushover analyses (see Table 1). The failure localized to wall 4 due to diagonal cracking at first storey, according to pushover prediction, at about 3‰ drift. Maximum absolute displacements at mid-span of top floors were always lower than 2% drift; therefore, out-of-plane failure of walls orthogonal to the seismic direction did not occur, also for the configuration with the original floors.

Figures 8 and 9 show the results from IDA at near-collapse PGA for all configurations. Results are given in terms of time-displacement curves between 8 and 13 sec., i.e., within a range including the peak displacements. The controlled points are B1, B2 and B3 in Figure 8 to compare the displacements of walls and floors; B1 and B3 in Figure 9 to compare displacements of wall 2 and 4. These Figures clearly show that the addition of 45° timber boards or steel plates reduces significantly the deformations of the floors, whereas the floors strengthened with the 50-mm thick RC slab underwent no deformations. However, the effects in the seismic response of the building are essentially the same for all three techniques and all the stiffened diaphragms can be assumed rigid in their plane. Conversely, the original floor acts as a deformable diaphragm and is not able to confer to shear walls the same displacements. It is worth noting that the peak horizontal displacement of the original floor is about two and a half times higher than that of the weakest wall (wall 4) at the middle floor. This difference increases to six times at the top floor. Such displacement values might lead to out-of-plane failure of the orthogonal masonry walls, but at the same time, it can be expected that the shear deformation of the floor could contribute to the dissipative capacity of the building, because relative displacements of the floors exceed their yielding limit (see Figure 5a).

Figure 10 shows the results from IDA for all the PGA levels analysed. The near-collapse PGA reached by the building with the original floors is about 1.2g. It decreases to about 0.8g and 0.75g for the configurations with 45° timber boards and steel plates respectively and falls to about 0.65g for the configuration with the RC slabs due also to the increased seismic mass of the building.

Contrary to the commonly accepted expectation and to results from pushover analyses, these results seem to demonstrate that the stiffening of original timber floors made with a single layer of timber boards actually leads to a reduced seismic capacity of masonry buildings, if first-mode failures are avoided by means of suitable methodologies. The explanation of this phenomenon is that the deformable floor acts as a dissipative damper interposed between the floor mass and the resisting shear walls if subjected to plastic deformations. The increased oscillation period and dissipation capacity produce a reduction of seismic forces on resisting walls.

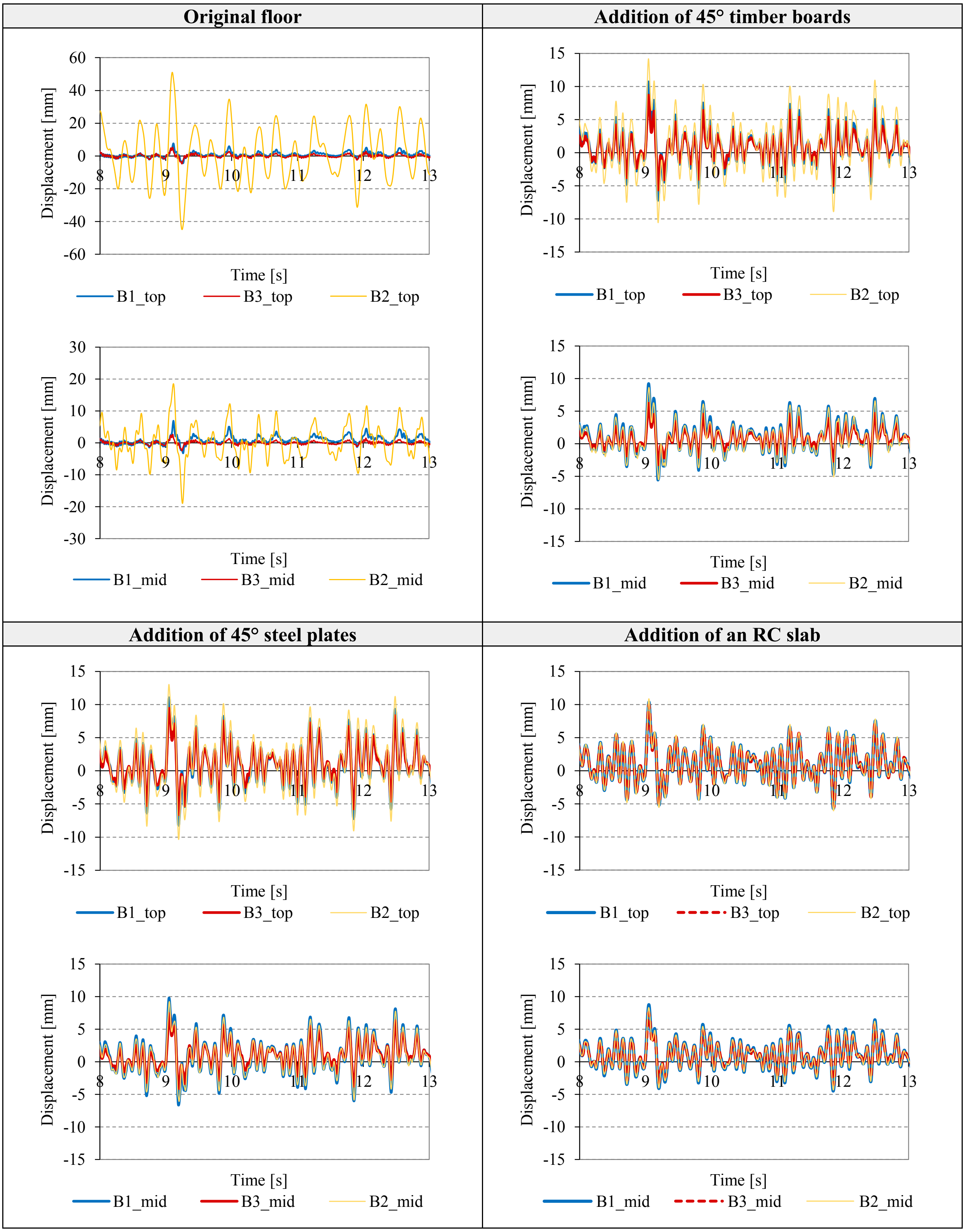


Figure 8: Results from time-history analyses at near-collapse PGA in terms of time-displacement curves between 8 and 13 sec. (including the peak displacements) for walls and floors.

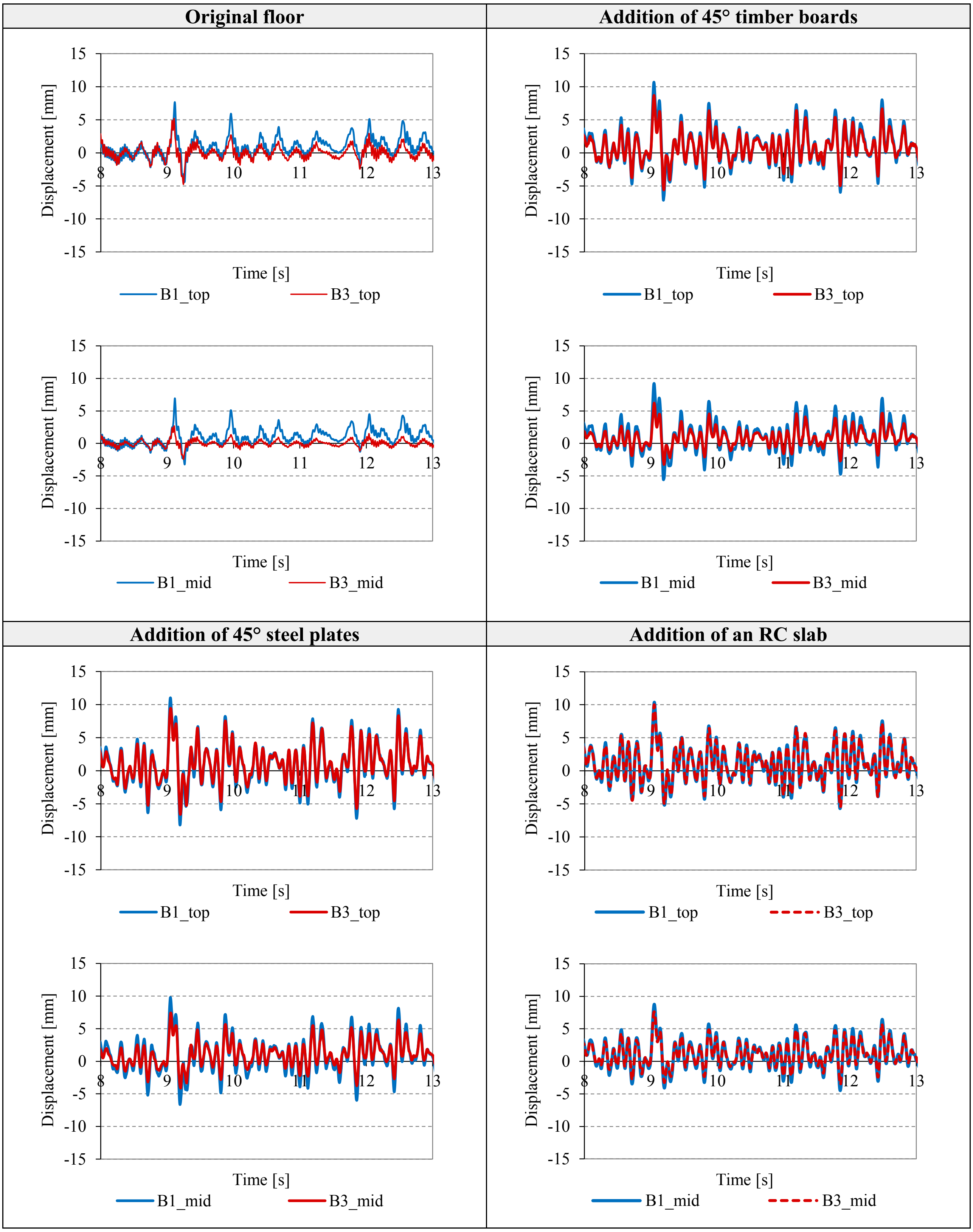


Figure 9: Results from time-history analyses at near-collapse PGA in terms of time-displacement curves between 8 and 13 sec. (including the peak displacements) for walls 2 and 4.

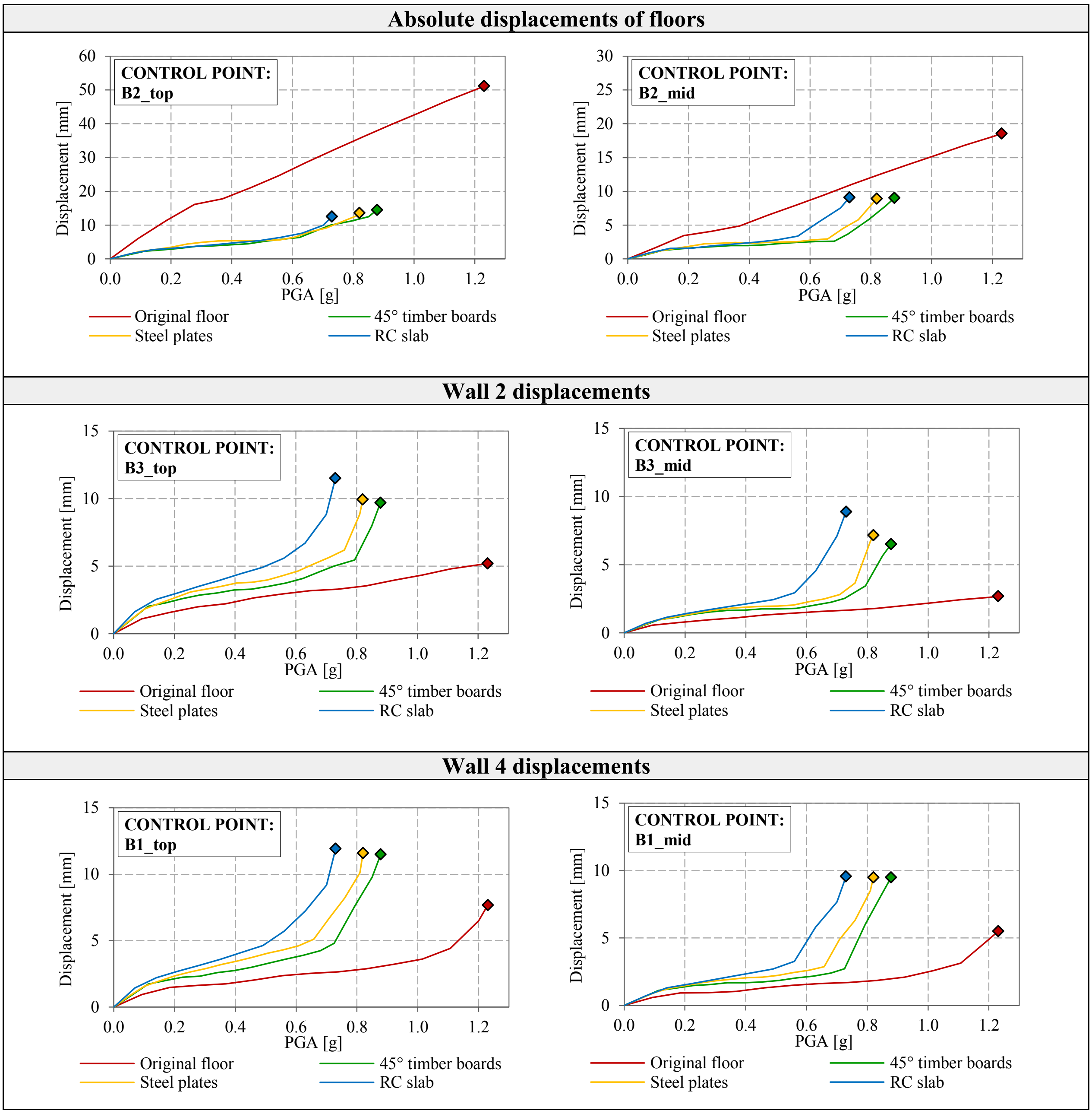


Figure 10: Results from IDA in terms of displacements at various control points at increasing PGA levels.

4 CONCLUSIONS

In this work, the modification of the seismic response of a two-storey masonry building subjected to different rehabilitation techniques of the timber floors has been analysed with non-linear pushover and incremental dynamic simulations. The chosen techniques consist in the addition of 45° timber boards, of steel plates or of a RC slab above the exiting floors.

Results from pushover analyses apparently said that the in-plane stiffening of timber floors improved the seismic performances of the building of about 40%. This improvement derives from the increased collaboration between shear walls to withstand the seismic action. However, pushover methods do not take into account of the different dissipative capacities of alternative floors, but only of their stiffness and strength. Therefore, such results could be distorted.

In fact, the incremental non-linear dynamic analyses showed oppositely that the seismic performance of the case-study building decreased of about 35% for configurations with 45° timber boards and steel plates and 45% for the configuration with the RC slab. This means that a retrofitting method leading to excessive in-plane stiffening of the floors worsens the actual seismic performance of a masonry building. This is in contrast with common belief. The reason is that a deformable floor can act as a dissipative damper interposed between the floor mass and the resisting shear walls. The increased oscillation period and dissipation capacity produce a strong reduction of seismic forces on resisting walls, without allowing excessive out-of-plane deformation of the wall orthogonal to seismic excitation. Obviously, ties and rigid connections between floors and walls are necessary to avoid 1-st mode overturning of orthogonal walls and to improve the transmission of shear forces between floor and resisting walls.

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