

Pushover analysis of traditional masonry buildings: influence of refurbished timber-floors stiffness

Ivan Giongo¹, Maurizio Piazza², Roberto Tomasi³

Abstract The main purpose of this paper is to evaluate the effects that in-plane stiffness of different types of wooden diaphragms, yield on the capacity curve of a traditional masonry building, obtained by means of a nonlinear static (pushover) analysis. In order to determine it, an equivalent-frame modelling has been employed to schematize a two-storey building, the like of which is fairly common in the Italian building legacy. Both as-built and retrofitted wood floor types have been taken into account. As to better understand and control all the aspects that rule the global seismic behaviour of a masonry construction, a general-purpose FEM software has been adopted. Therefore every "feature" has been manually implemented.

Keywords in-plane stiffness, pushover analysis, wooden floors, masonry building, equivalent frame

1. INTRODUCTION

It is utterly acknowledged (with no need to mention the effects of recent seismic phenomena) how important it is for structural designers to know and master a reliable, simplified method able to analyze the global seismic response of masonry buildings. Several studies have shown that masonry is highly nonlinear, even for low levels of stress. As a result, linear elastic methods are not fit to represent it. Recently, new techniques are getting more and more widespread, i.e. the so-called macro-elements methods. The main advantage of these methods, with respect to techniques based on shell or solid nonlinear finite-elements, is the decrease in run-time analysis due to the concentration of all the nonlinearities in some specific points. Furthermore, referring to equivalent-frame structures allows designers to deal with simplified constitutive laws and failure criteria. In order to "capture" the real global behaviour of a traditional building, it is necessary to optimize these numerical models through parametric analyses which investigate numerous aspects that might affect the seismic performance, from frames geometry to lateral loads distribution etc.. Once one has defined how to schematize the masonry skeleton he/she needs to model horizontal diaphragms, whose in-plane stiffness plays an undeniable key-role in distributing seismic lateral loads to the resisting walls. As a matter of fact it is expected that the more the in-plane stiffness grows, the more the collaboration between systems of

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piers increases. In addition, earthquake damages have demonstrated that the in-plane stiffness of horizontal diaphragms often influences the out-of-plane walls response, by determining the type of local mechanism occurring (II mode mechanisms). Nevertheless, as this paper regards the global seismic performance, no results have been reported concerning this matter.

2. THE MODELLING

2.1. The building

The numerical model used to investigate the effects that the real in-plane stiffness of wooden floors yields on seismic behaviour of masonry buildings, is based on the so-called "equivalent frame" method. Consequently every pier and every spandrel is schematized with an elastic frame element. The mechanical nonlinearities are concentrated in particular cross-sections (plastic hinges) placed both in the middle and in the ends of the elastic frames. Since the analysis of real "post-earthquake" masonry buildings has shown that most of the damages do not involve the intersections between piers and spandrels, rigid offsets are inserted where the vertical elements meet the horizontal ones. The length of these offsets depends on the geometry of the openings (windows and doors). In particular the effective height of a pier (correspondent to the elastic part of the frame) has been deduced from a formula developed by Dolce in 1989 (Dolce 1989). Referring to the in-plane behaviour of walls, the bottom ends of the vertical frames (piers) have been modelled as fixed (FEMA 356). On the other hand, considering the global seismic performance of masonry buildings, the out-of plane stiffness of the walls has been regarded as negligible and therefore moment releases have been introduced at both ends of the piers (FEMA 356).

2.1.1. Piers

According to Magenes and Calvi (Magenes and Calvi 1997), three different failure criteria have been considered: rocking, sliding and shear cracking. The following formulae have been used:

- Rocking:

$$M_u = \frac{P \cdot D}{2} \left(1 - \frac{p}{k f_u} \right) \quad (1)$$

where M_u = moment of resistance, P = axial load (concentrated), D = pier length, $p = P/Dt$, t = pier thickness, f_u = compressive strength of masonry, k = parameter depending on the stress distribution assumed at the pier base ($k = 0.85$). Equation (1), is the same proposed in the Italian Code NTC 2008.

- Sliding:

$$V_d = Dt \left(\frac{1.5c + \mu p}{1 + 3 \frac{c \alpha_v}{p}} \right) \quad (2)$$

where V_d = "the ultimate load of a wall" (Magenes and Calvi 1997), c, μ = mechanical parameters related to the Mohr-Coulomb criterion, $\alpha_v = M/VD$ (*shear ratio*), M, V = moment and shear forces acting on the pier.

- Shear cracking:

$$V_u = \frac{f_{tu} \cdot D \cdot t}{b} \sqrt{1 + \frac{p}{f_u}} \quad (3)$$

where $V_u = V_d$, f_{tu} = tensile strength of masonry, b = coefficient depending on the shear distribution at the central cross-section of the pier ($b = 1$ for uniform shear distribution, $b = 1.5$ for parabolic distribution). The tensile strength f_{tu} is assumed to be equal to 1.5 times the mean shear strength of masonry f_{mvo} (under zero compressive stress). Equation (3) is consistent with the formula suggested by the former Italian code O.P.C.M. 3431 for existing buildings.

In accordance with the suggestions contained in D.M. 14-01-2008 an elastic-perfectly plastic law has been assumed for both flexural and shear behaviour. The post-elastic state has been modelled by means of rigid-perfectly plastic hinges (Pasticier et al. 2007). A "rocking hinge" has been inserted at each end of the elastic part of the frames (their activation occur when moments acting on these extremities reach M_u), with no limits on deformation. Although the Italian Code (D.M. 14-01-2008) prescribes an ultimate rotation corresponding to a lateral deflection of 0.8% of the height of the pier,

an indefinitely plastic behaviour has been preferred. This choice is due to two reasons. First of all one should determine *a priori* which quota of the ultimate lateral deflection has to be assigned to each hinge. Moreover, this presupposes that the ratio between the bending moments acting on the ends of the elastic part of the walls is known at every step. Secondly, a lateral deflection of 0.8% of the height of the piers, means a displacement greater than 20 mm (for walls higher than 2.5 m) which is unlikely to be attained. As a matter of fact, the same Code proposes that in case of shear mechanisms, the maximum lateral deflection has to be limited to 0.4% of the height of the pier. This implies that, for the case study analysed (two-storey building), when one wall reaches the target of 0.8%, the total base shear has already diminished of more than 20% (owing to shear failure) and therefore (D.M. 14-01-2008) the pushover analysis has already been stopped. A shear hinge has also been introduced at the mid-span of the frames (as the external loads have been applied to the nodes of the frame, the shear diagrams are uniform). The hinge is activated by the minimum value between $V_{d,top}$, $V_{d,bottom}$, and V_u ($V_{d,top}/V_{d,bottom}$ corresponds to V_d with α_v calculated at the top/bottom section of the elastic part of the pier). According to the Italian Code, the shear plastic phase has been limited to a maximum drift of 0.4% of the pier-height.

The values of p and α_v , maintained constant during the analyses, have been deduced from a linear static procedure.

2.1.2. Spandrels

Two "rocking hinges" without any limits to deformation have been added to the elastic ends of the spandrels. The ultimate moment M_u that activates the hinges, has been determined as suggested in NTC 2008:

$$M_u = H_p \frac{h}{2} [1 - H_p / (0,85ht)] \quad (4)$$

where $H_p = 0,4f_{hm}ht$, f_{hm} = mean compressive strength of masonry in the horizontal direction, h = height of the spandrel, t = thickness of the spandrel. Like for the piers, a "shear hinge" has been inserted at the centre of the frames schematizing the spandrels. This hinge (indefinitely plastic) is activated when the shear force reaches the minimum value between the following two criteria:

$$V_p = \frac{2M_u}{l} \quad (5)$$

$$V_t = h \cdot t \cdot f_{mv0} \quad (6)$$

where f_{mv0} = the mean shear strength of masonry (under zero compressive stress), l = length of the spandrel.

2.1.3. Reinforced Concrete Stringcourses

Reinforced concrete stringcourses have been modelled at the top of every level, with a bi-linear elastic perfectly-plastic constitutive law. As like piers and spandrels, so RC beams have been schematized through elastic frames with rigid offsets at the extremities. Two "rocking hinges" (rigid, perfectly-plastic, without any restrictions on deformation) have consequently been added at the ends of the elastic part of the beams. The failure criterion has been deduced from European Standard: Eurocode 2 (UNI EN 1992-1-1).

2.2. The wooden floors

In order to study the influence that the in-plane stiffness of wood diaphragms has on the global behaviour of a masonry building under seismic condition, different types of unreinforced/reinforced floors have been modelled. The mechanical properties of floors have been derived from an experimental campaign previously carried out at the Laboratory of the Department of Mechanical and Structural Engineering (DIMS) of the University of Trento (Piazza et al. 2008, Baldessari et al. 2009). Only the membrane behaviour of floors has been taken into account, by means of nonlinear, two-dimensional finite elements. Considering the test set-up (Figure 3), an equivalent shear stiffness G_{eq} has been calculated from the experimental data, regarding the diaphragm deformation as equal to the shear deformation of a simply supported beam under a uniform load distribution.

$$G_{eq} = \frac{\chi \Delta W \cdot L}{8 \cdot B \cdot t \cdot \Delta f} \quad (7)$$

where χ = the shear factor, L = floor span perpendicular to the load direction, B = floor span parallel to the load direction, t = floor (membrane) thickness, ΔW = lateral load applied [N], Δf = mid span deflection.

In accordance with the Theory of Elasticity, a nonlinear relation between shear stress and shear strain has been directly derived from the G_{eq} curves.

3. THE EXPERIMENTAL DATA

The experimental data pertain to a campaign conducted on six different types of wooden diaphragms (Figure 1). Both monotonic and cyclic tests were been carried out on full scale specimens (5×4 m). The overall test set-up is shown in Figure 3, while in Figure 4 the experimental results are reported in terms of a total load Vs. mid-span displacement curve, for every type of tested floor. Every shear stress/strain law has been validated through FEM models of the tested specimens, based on nonlinear shell elements (with just membrane behaviour). In Figure 2 it is depicted, e.g. for the double floor, the good agreement between experimental and numerical behaviour (external load Vs. mid-span displacement).

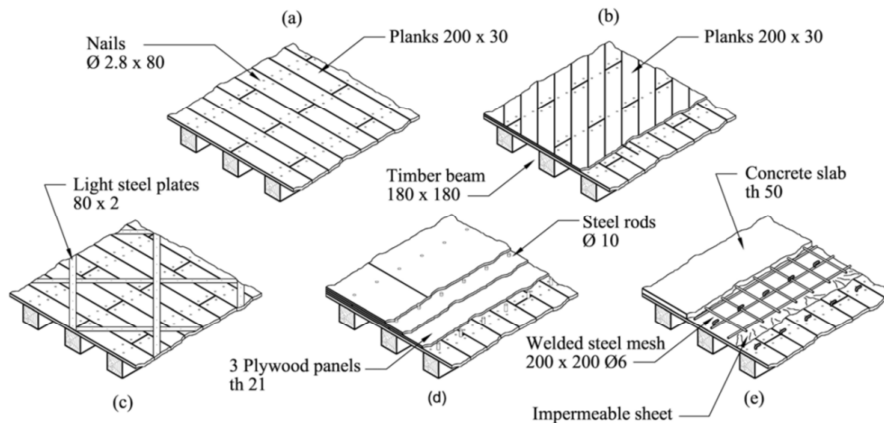


Figure 1 - Different timber-floor in-plane-shear strengthening techniques: (a) existing simple layer of wood planks on the timber beams; (b) second layer of wood planks crossly arranged to the existing one and fixed by means of steel studs; (c) diagonal bracing of the existing wood planks by means of light steel plates or FRP laminae; (d) three layers of plywood panels glued on the existing wood planks; (e) a stud-connected reinforced concrete slab (all measures in mm).

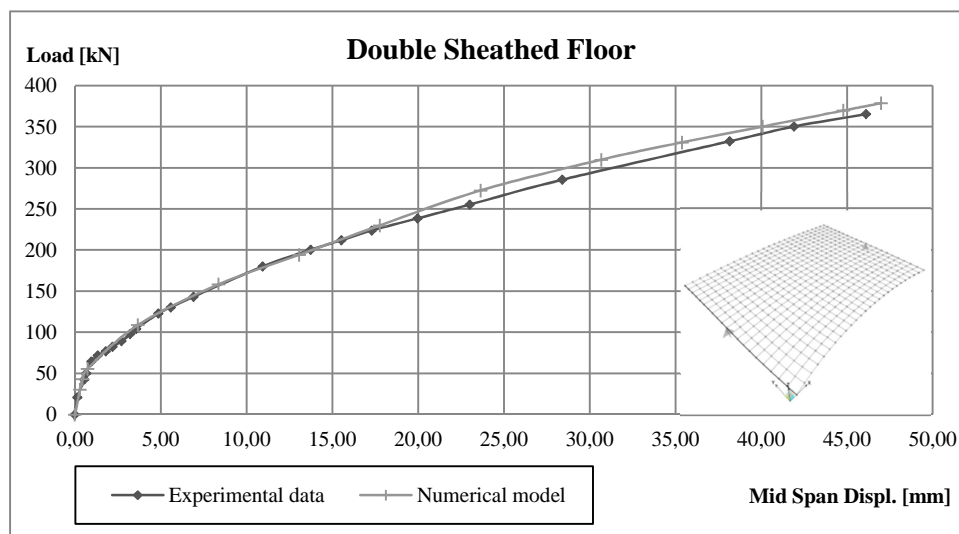


Figure 2 - Validation of numerical floor-models

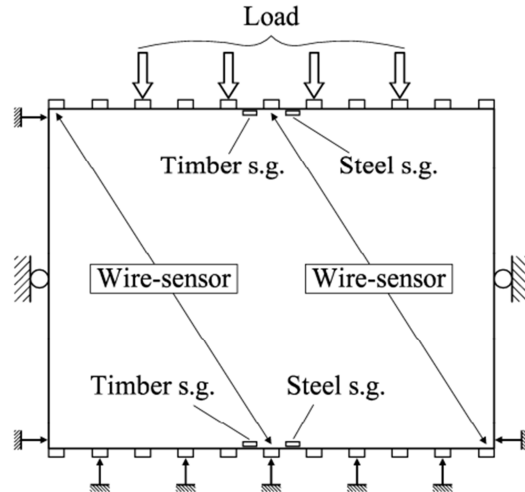


Figure 3 - The test apparatus

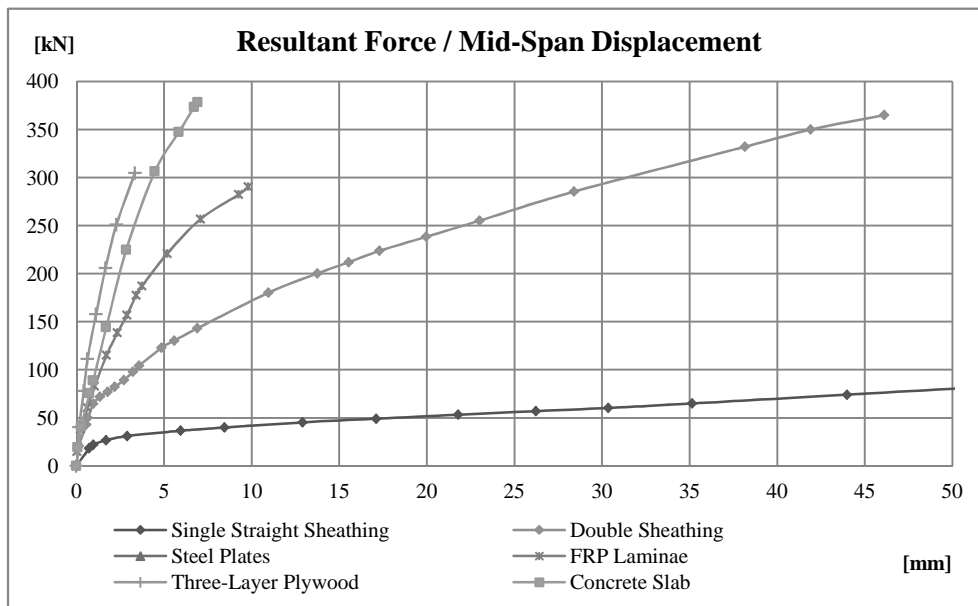


Figure 4 - In-plane behaviour of tested floors

4. CASE STUDY BUILDING

The masonry building selected for the parametric analyses has been found in literature (Righetti and Bari 1993). The choice has been prompted by the need of analyzing a masonry construction neither extremely irregular (the results would have been too dependent on the specific structure studied), nor particularly regular (as not to be too dissimilar from "real buildings"). Furthermore, since the structure is a two-storey building, it is possible not to consider the variation in axial forces that the development of the pushover analysis yields; as suggested by the Italian Code in its former version (OPCM 3431).

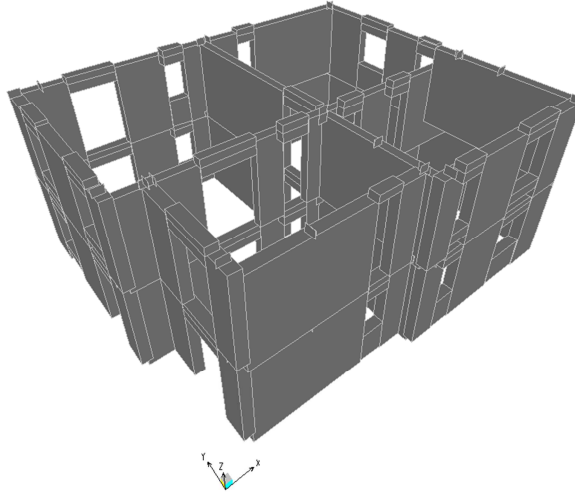


Figure 5 - Analyzed building

Table 1 - Mechanical properties of masonry

Weight density of masonry	γ_m	18	kN/m^3
Characteristic compressive strength of a brick	f_{bk}	10	MPa
Characteristic compressive strength of masonry	f_k	4.5	MPa
Characteristic shear strength of masonry (under zero compressive stress)	f_{vk0}	0.2	MPa
Elastic modulus of masonry	E	4500	MPa
Shear modulus of masonry	G	1800	MPa
Mean compressive strength of masonry	f_m	6.5	MPa
Mean shear strength of masonry (under zero compressive stress)	f_{vm0}	0.3	MPa
Mean compressive strength of masonry (horizontal direction)	f_{hm}	2.0	MPa
Friction parameter	μ	0.4	

Table 2 - Mechanical properties of concrete

Weight density of concrete	γ_c	24	kN/m^3
Characteristic compressive strength of concrete	R_{ck}	30	MPa
Elastic modulus of concrete	E	30	GPa

5. THE ANALYSES

5.1. The parametric analyses

Together with the in-plane stiffness of wooden floors, many other parameters that affect the global seismic behaviour of a traditional building, have been analyzed. As to evaluate the influence of spatial variability of the seismic ground motions, the mass centre has been moved into different positions ($\Delta x = \pm 0.54 \text{ m}$, $\Delta y = \pm 0.66 \text{ m}$) as counselled by the Italian Code. No differences have been observed in the static pushover curves related to different positions of the mass centre (hypothesis of rigid diaphragm).

Two different lateral load patterns have also been applied during the pushover analyses: an inverted triangle (first mode) load pattern and a mass proportional one. While in the y direction the two patterns have produced no effects on the capacity curve, in the x direction the maximum base shear shows a difference greater than 30%. As one can see from Figure 6, the mass proportional pattern stresses the first storey more than the other (there are more active hinges), delaying the formation of plastic hinges in the upper storey. This, considering that the global failure (x direction) is always a second-storey mechanism, generates an increase in resistant base shear. It would appear that the following sentence contained in the ASCE Standard (ASCE/SEI 41-06): "*Recent research [FEMA 440 (FEMA 2005)] has*

shown that multiple load patterns do little to improve the accuracy of nonlinear static procedures and that a single pattern based on the first mode shape is recommended" does not fit the studied building.

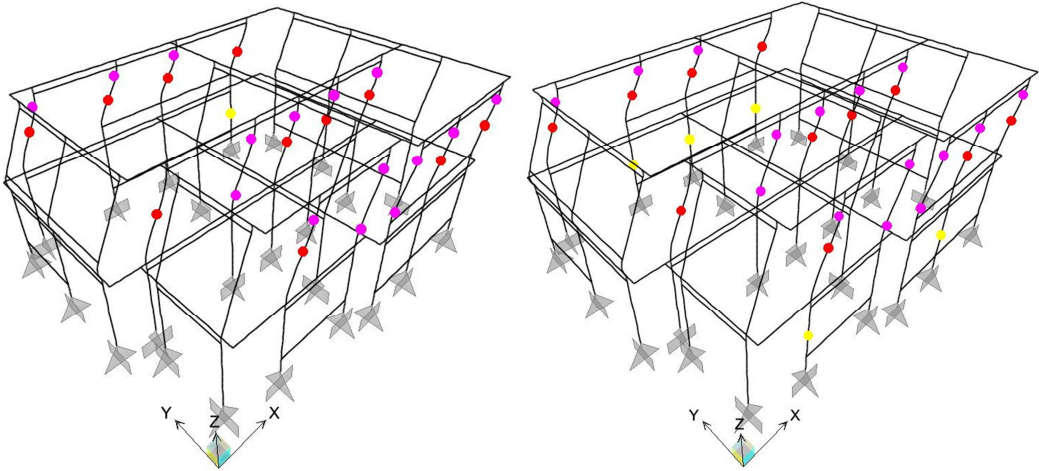


Figure 6 - X direction deformed shape - (Inverted triangular load pattern on the left; mass proportional load pattern on the right) - Plastic hinges: purple = rocking hinge, red = hinge that has reached the deformation limit, yellow = bottom shear hinge, orange = top shear hinge.

In order to determine the effects that the stiffness of the stringcourses induces on the seismic performance of an existing building, the elastic modulus of concrete has been varied, from the one of un-cracked material (30 GPa) to zero (the MOE used in all the other models has been 15 GPa, corresponding to the cracked material). Neither the base shear, nor the control point displacement have shown appreciable sensitivity to stringcourses stiffness. Only a predictable reduction in "global elastic stiffness" has been observed as the MOE value has decreased (Figure 7).

Another aspect to be stressed is the different "plastic demand" related to the spandrels. When concrete stiffness is that of cracked material, the global failure occurs with all the spandrels in the elastic phase. On the other hand (as expected) if one neglects stringcourses stiffness, "rocking hinges" are activated at the ends of some spandrels. On the contrary, regarding stringcourses as un-cracked, yields the activation of plastic hinges in the stringcourses themselves.

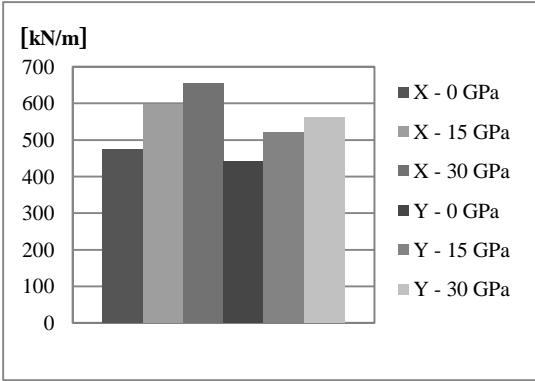


Figure 7 - Global secant elastic stiffness vs. concrete MOE

It has also been studied what happens when the coupling actions of both spandrels and stringcourses is not taken into account and the masonry walls are modelled as cantilever beams (Figure 8). These simplifying assumptions are frequently employed in Linear Static Procedure by designers and it is commonly believed that they lead to a higher level of safety than that attainable with a frame model. From a "resistance point of view" this is certainly true, in fact (as one might expect) a decrease of more than 35% in maximum base shear has been observed in both directions. Nevertheless, the application of a Non Linear Static Procedures requires a displacement check. Since the "cantilever

beams" hypothesis produces an increase of almost 200% in ultimate displacement of control point (Figure 9), it would appear that disregarding the coupling effect of spandrels could induce to overestimate the ductility resources of a masonry building.

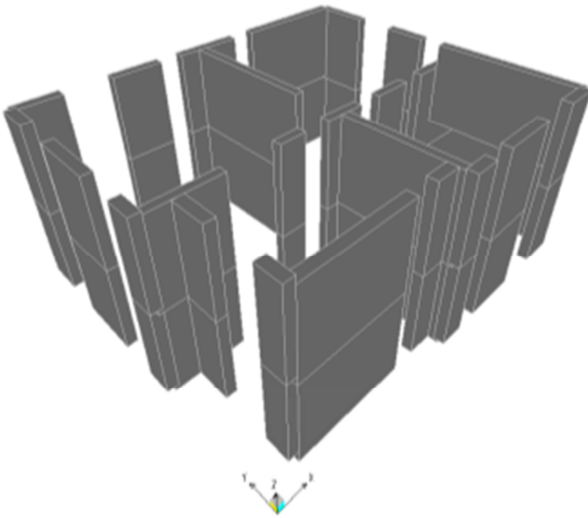


Figure 8 - Cantilever beams model (soft spandrel hypothesis)

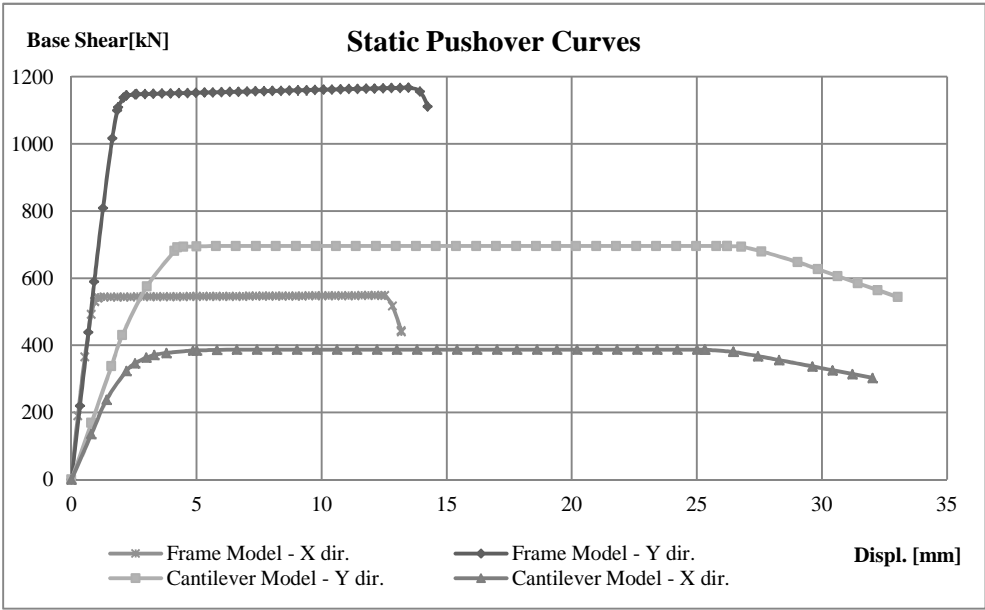


Figure 9 - Comparison between the seismic performance of equivalent-frame model and cantilever-beams model

Another aspect subjected to investigation has been the possible birth of fictitious internal forces, owing to the rigid links that connect orthogonal systems of walls. So as to seek the effects of these links, a model of the building with none of them has been created (rigid diaphragm hypothesis, Figure 11). From the results (Figure 12) it is possible to see that in y direction the removal of the links, reduces both the global elastic stiffness and the maximum base shear. As one might expect, it would appear that those reductions are due to a less restrained building. Nonetheless, in x direction, an increase in total shear resistance has been observed. A reason may be that if one eliminates the coupling action generated by the links, then the bending stresses at the top of the piers grow (under the same level of lateral loads); this leads to an extension of the effective length of compression, strictly related to a variation of α_v , that makes the shear resistance ($V_{d,top}$) raise. Since the global failure

mechanism, in x direction, is associated with the formation of shear hinges at the top of the second-storey piers, an increase in maximum base shear should therefore be considered plausible.

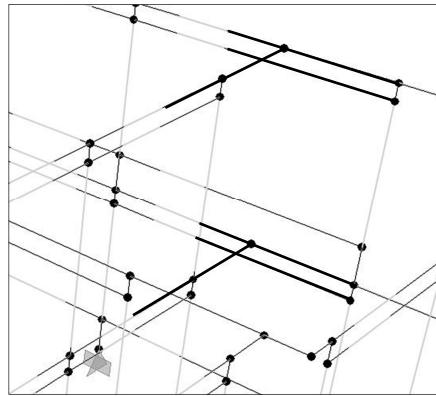


Figure 10- Rigid links connecting orthogonal systems of walls

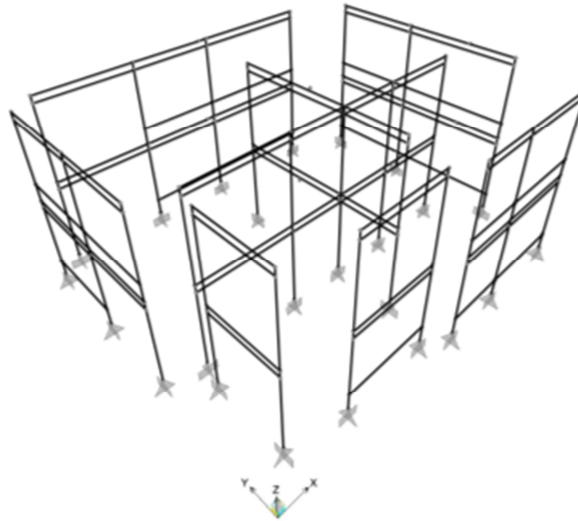


Figure 11 - Model without any link between perpendicular systems of walls

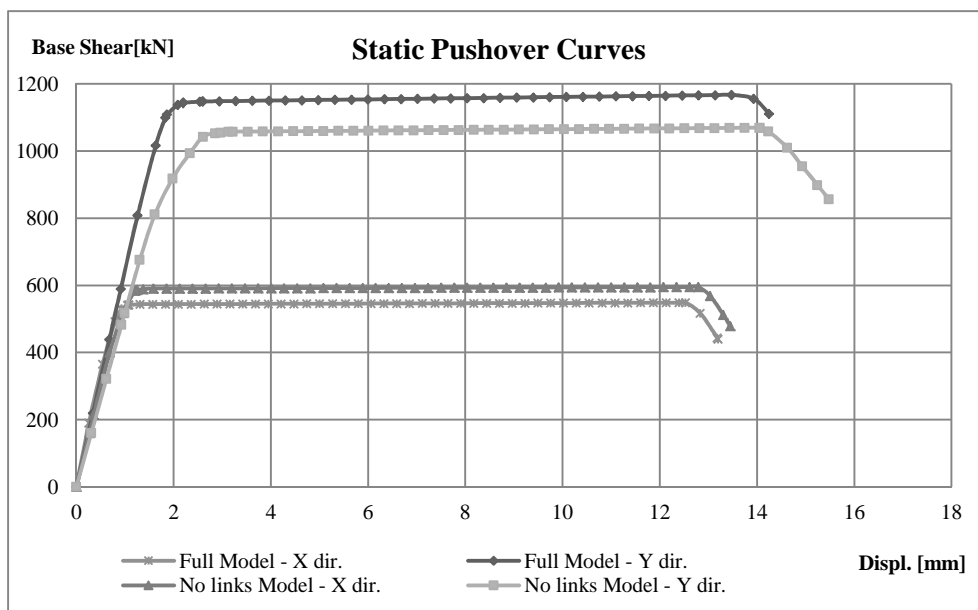


Figure 12 - Comparison between full model and no-links model

5.2. Influence of diaphragms stiffness

In Figure 13 and Figure 14 the results of static pushover analyses with the different wooden floors are reported. It is quite evident that, in terms of displacement capacity, the most common as-built traditional floor (single straight-sheathing) embodies a poor solution for a masonry building placed in a seismic zone (even though the tested one was very well-made, far beyond the common standard). Furthermore one can see that modelling single straight-sheathed floors with rigid diaphragms leads to overestimate both the shear resistance and the ultimate displacement of approximately 20% and 100% respectively. The reason is that rigid floors are capable of involving in the failure mechanism almost all the walls (storey mechanism). Actually when the building reaches the collapse point (80% of the shear resistance) nearly all the piers, directed along the earthquake direction, have already gone beyond the plastic threshold (Figure 16). On the other hand, a quite low in-plane stiffness means that there little "collaboration" between the systems of walls. That is to say, the failure occurs when a single system of walls attains its ultimate displacement (Figure 15). As a result, the analysis ends with many of the piers in the elastic phase, preventing the structure from obtaining the performance observed under the rigid diaphragms hypothesis.

Apart from the single straight-sheathed floor (whose in-plane stiffness is almost negligible), it appears that the maximum base shear is not affected by variation of the in-plane stiffness of diaphragms. In addition, in both directions the global stiffness does not show appreciable differences changing the type of floor refurbishment.

When the earthquake action is directed along y axis, the ultimate displacement of the pushover curve seems to be highly sensitive to the floor stiffness (Figure 14). As a matter of fact, the stiffer the floors is, the bigger the displacement capacity becomes. Nevertheless, with the rigid-diaphragms hypothesis the ultimate displacement is smaller than that obtained considering any other type of refurbished floor. A possible reason may be found analyzing the deformed shape of the structure. It has been observed that there is a sort of torsion movement that, starting from a certain point onwards (corresponding to the activation of a rocking hinge on a specific pier), significantly raises the stress level of the external walls (directed perpendicularly to the seismic action), leading the structure to failure. This "trigger point" has proved to be postponed if the floors stiffness increases and, consequently, the analysis can go further too. On the other hand if this torsion movement is entirely stopped (rigid-floors hypothesis) many more piers reach their ultimate displacement simultaneously (shear failure), calling the analysis to a premature halt.

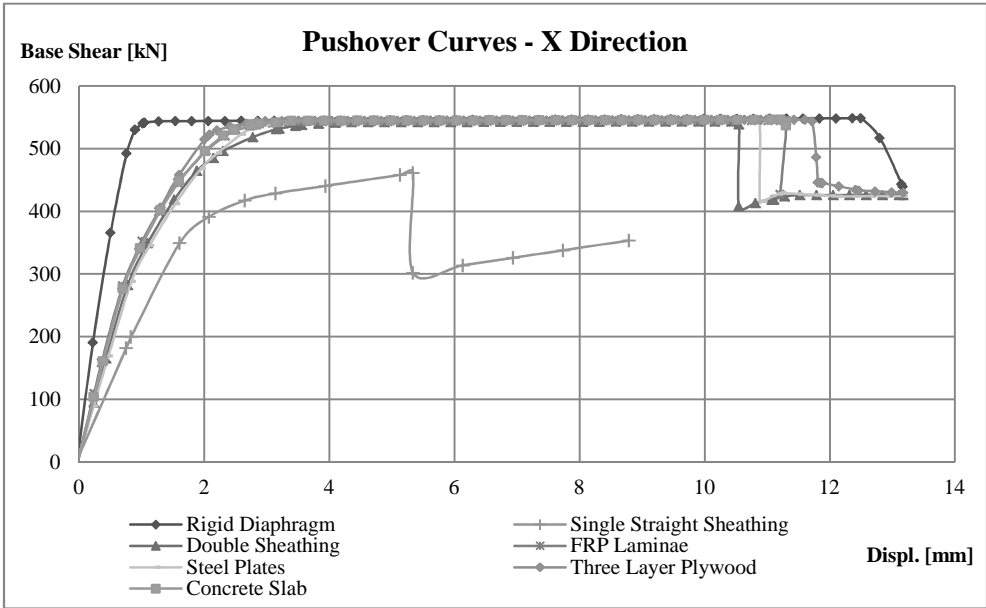


Figure 13 - Nonlinear static analyses: capacity curves

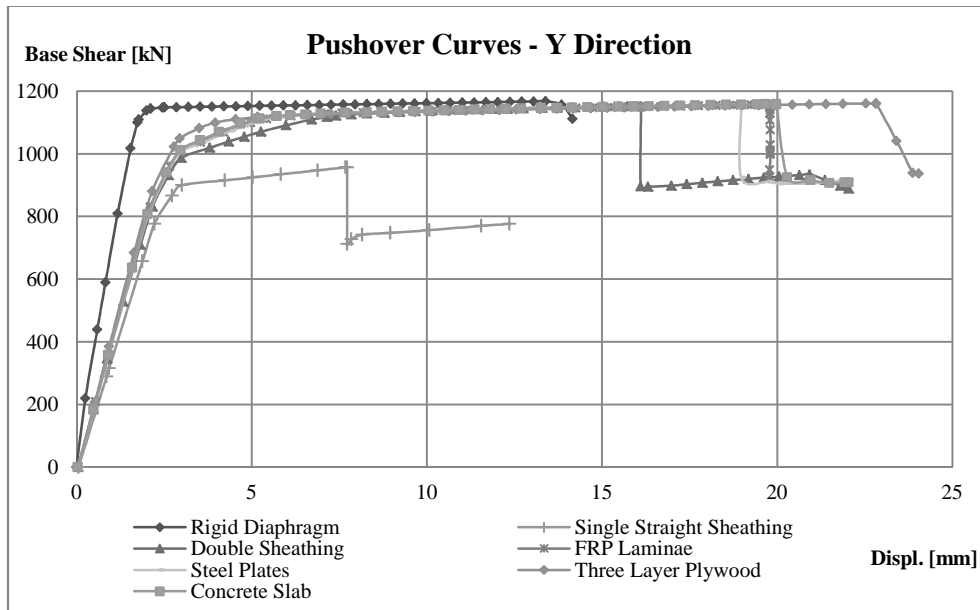


Figure 14 - Nonlinear static analyses: capacity curves

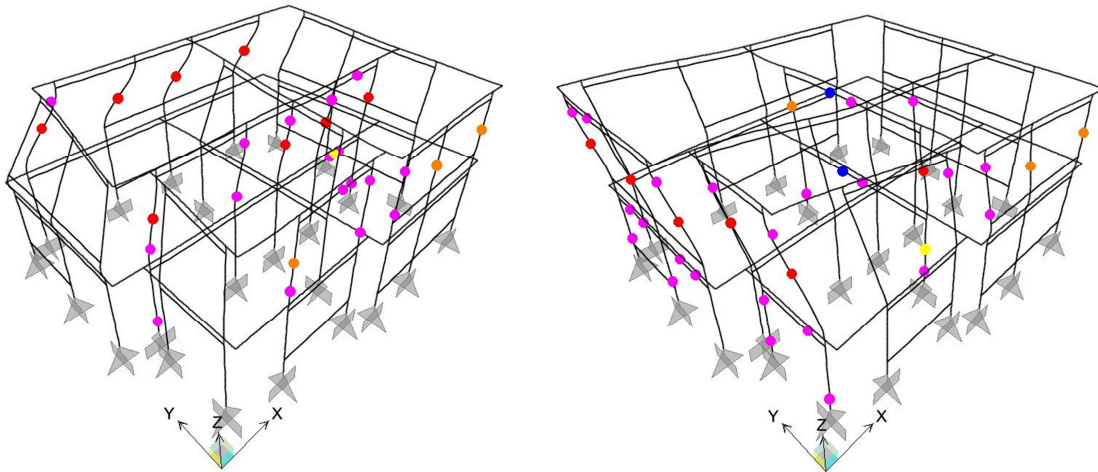


Figure 15 - Deformed shape (single straight-sheathing hypothesis): x dir. on the left, y dir. on the right

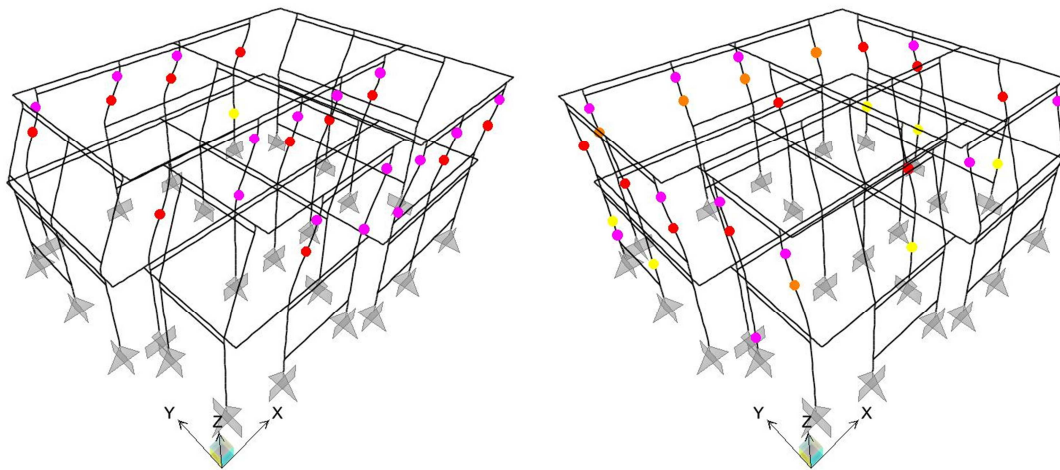


Figure 16 - Deformed shape (rigid diaphragm hypothesis): x dir. on the left, y dir. on the right

6. CONCLUSIONS

From the presented results it would appear that a floor strengthening and stiffening is quite basic to the improvement of the seismic response of a masonry construction. Yet, the choice of the type of refurbishment does not seem to be of any relevance at all. Nonetheless, before leap to conclusion one should consider that the capacity curve is strictly linked to the building geometry and to the load distribution. Moreover, although the global behaviour could be not affected by the in-plane floor stiffness, not so is the local one.

An interesting aspect that ought to be more deeply studied, is the possibility of updating failure criteria of piers by determining step by step the values of p and α_v .

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