# In-plane behaviour of innovative masonry infills based on different configurations of wooden sliding joints

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## Abstract

The seismic vulnerability of masonry infilled frames represents a critical issue in reinforced concrete and steel buildings, widely highlighted by ruinous collapses during earthquakes and studied by many authors. In these structures, the interaction between the flexible frame and the rigid masonry infill modifies the dynamic response of the frame, inducing possible undesired collapse mechanism, and, on the other hand, can cause widespread damage in the infills. To avoid these detrimental effects, a technological solution for the design of the infills has been studied and tested in the last years at the University of Brescia, consisting in partitioning the masonry with wooden planks, working as sliding joints. Tests have shown the potential of the solution for providing a superior performance than traditional masonry infills, thanks to the reduction of the detrimental effects of the infill frame interaction. The benefit comes from a significant reduction of the infill strength and stiffness, a limitation of its damage under in-plane loading and a ductile behaviour, with energy dissipation capacity that can be easily predicted. In this paper a numerical study that extends test results is presented. A parametric analysis of the response of the infills has been performed as a function of some geometrical and mechanical properties: stiffness and strength of the materials, dimension of the infill and configuration of the sliding joints. The results offer information necessary for the application of the construction technique, which is proposed to improve the infilled frame seismic response.
*Keywords: masonry infill, sliding joints, material testing, numerical modelling.*

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# 1 Introduction

The unfavorable interaction of the RC frames with the rigid infill walls caused many building collapses in recent earthquakes (e.g. L'Aquila 2009 and Kocaeli 1999). Typically, seismic response of the infill wall is ignored in the design, even if its interaction with the frame significantly affects the behavior of the structures, making it difficult to predict the actual collapse mechanism and increasing the risk of developing an undesirable weak-story or short-column mechanism in the frame structure. A number of construction techniques have been proposed and investigated in order to mitigate this problem. Such techniques typically aim at increasing the infill stiffness and strength to prevent the structure's collapse during an earthquake [1–3]. Although such methods increase the seismic capacity of infilled frames, they can influence its dynamic behavior, modifying the response with respect to the widely adopted assumption of a ductile mechanism with hinges on the frame beams and at the base of its columns. As a consequence, particular care has to be paid to the evaluation of the collapse mechanism of the structure in order to avoid possible brittle failures.

A construction technique for the infill, based on an alternative design approach, was developed to allow a reliable prediction of the structural performance in terms of strength and ductility, controlling the interaction between the infill and the surrounding frame [5–8]. In the tests conducted by Preti *et al*. [6], Migliorati [7] and Stavridis and Shing [8] a stable frictional sliding mechanism of the infill was ensured thanks to the introduction of a number of wooden boards partitioning the masonry wall in subportions. Wooden boards, placed horizontally in some mortar beds, create weak surfaces, along which the deformation of the infill concentrates in the form of relative sliding of the infill subportions. In addition, the introduction of vertical wooden boards (weak elements) at the columns-infill interface and the presence of a gap between the wall and the top beam allowed large deformations without strain concentration and consequent damage in the masonry.

In this paper, the calibration of a numerical model [9] is described, starting from the infill component material properties, capable to reproduce the results experimentally obtained in [6–8]. By means of this numerical model, a parametric study on the influence of mechanical and geometrical parameter on the response of the infill has been carried out. In particular the role of the material strength and stiffness, of the infill geometry and of the joints configuration have been investigated to obtain some design criteria useful to ensure the efficiency of the proposed technological solution.

For the model calibration, some material characterization tests were carried out, whose set-up and results are presented in the paper.

# 2 Numerical model

The behaviour of two tested infills [7, 8] has been simulated with finite element models combining discrete and smeared-crack elements. The elements have been developed by Lotfi and Shing [9] and implemented in the program FEAP. The steel frame has been modelled with elastic elements, while the masonry infill panels have been modelled with smeared-crack and interface elements as proposed in [10]. The constitutive models for the finite element analyses have been calibrated with a methodology proposed by Stavridis and Shing [8], using data from material tests conducted simultaneously with the tests on infills, as well as information from the literature.

# 3 Material mechanical characterization

The mechanical properties of the materials used in the experimental specimens have been determined by performing tests on the single infill components and masonry assemblies.

## 3.1 Test on masonry

In order to evaluate the compressive strength and modulus of elasticity of the hollow clay masonry, compression tests on small masonry prisms, in parallel and perpendicular direction to the brick holes, have been carried out (as shown in Fig. 1).

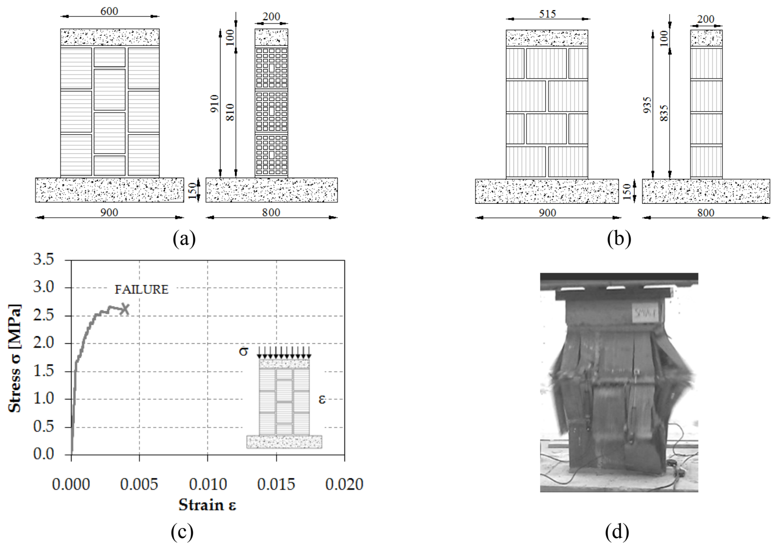


Figure 1: Details of tested assembly loaded in direction parallel to the holes (a) and perpendicular to the holes (b). Stress-strain curve for compressive test on a masonry assembly in the direction perpendicular to the holes (c) and its failure (d).

Fig. 1(c) shows the stress-strain curve for one of the tests on a wallet loaded in direction parallel to the holes and in Fig. 1(d) a picture of its brittle failure is reported. The average values obtained are $f_{m,c//}$=7.28MPa and $E_{m//}$=16148MPa for the wallet loaded parallel to the holes, while $f_{m,c\perp}$=2.40MPa and $E_{m\perp}$=4408MPa in direction perpendicular to the holes.

Tests on masonry triplets have been carried out to evaluate the shear resistance along the mortar joints. The test set-up is shown in Fig. 2: the shear load has been applied to the central brick by means of a jack and transferred to the specimen through a hinged connections (2 and 3 in Fig. 2(a)) to apply the vertical load aligned with the shear surfaces in the mortar. The lateral blocks of the specimen were simply supported by a HEB 220 (1 in Fig. 2(a)). A transversal precompression has been applied to the specimen by means of two threaded rod (5 in Fig. 2(a)) with soft elastic springs (8 in Fig. 2(a)) which helped in maintaining constant of the confining stress, compensating its variability due to the dilatation of the specimen. The specimens have been tested with 4 different levels of precompression to quantify the peak and residual sliding shear strength for different normal stress conditions.

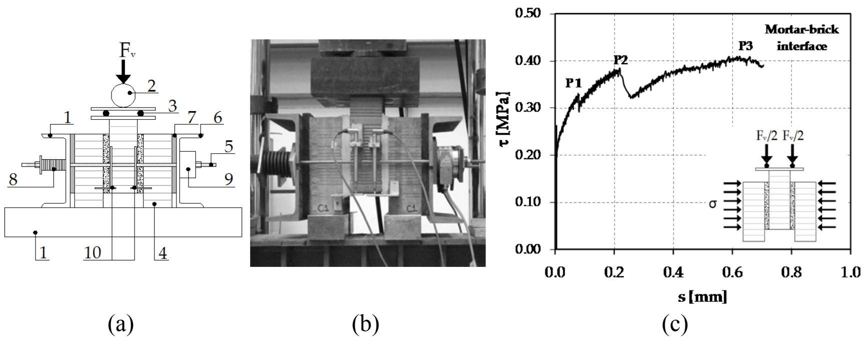


Figure 2: Shear test: test set-up (a, b) and typical shear stress–strain curve of mortar-brick interface (c), taken from one of the tested specimens.

Fig. 2(c) shows a typical shear stress-strain curve obtained for the specimens. Analyzing the graph, the activated shear mechanism is characterized by several peaks in the response. In fact, the first peak is related to the maximum cohesive mortar-brick interface shear strength; in the test a higher shear capacity has been registered after sliding occurred, related to the "dowel action" acted by the mortar "teeth" penetrating the holes of the brick and supported by the brick inner webs. The following drops in the response are typically related with such webs or teeth failure. In Fig. 3(b) are presented the average results obtained, in terms of first peak shear-strength, varying the normal stress. The same test has been carried out to evaluate the shear behavior of the sliding joints made of a wooden board immersed in the bed joints between masonry blocks. In this case the tested specimen consisted of two wooden boards with a block interposed (Fig. 3). On the brick-wood interface a polyethylene sheet has been also introduced.

### 3.2 Test on wood

The wooden boards adopted in the tested experimental wall have been tested in compression to evaluate their strength and stiffness in direction perpendicular to the wood fibers. Five specimens with dimensions of 50x40x25mm have been tested showing an elastoplastic behaviour with an average strength equal to 2.56MPa and an average elastic stiffness of 255MPa.

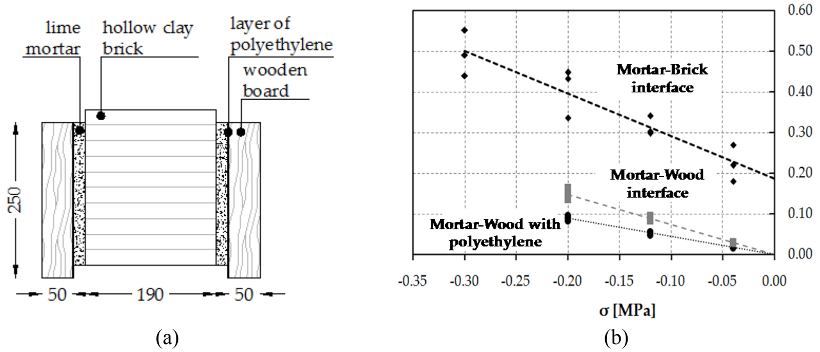


Figure 3: Sliding joint interface specimen (a) and results of the tests on triplets for the different interfaces considered (b).

# 4 Calibration of the model and comparison with the experiments

The parameters required to define the materials in the numerical model have been calibrated partially on the basis of the local characterization tests and partially on data collected in literature. All the parameters adopted for the numerical model (Table 1) are also the result of an a-posteriori fitting carried out on the results of the tested continuous infill (without sliding joints) in order to better capture the overall response of the masonry.

## 4.1 Calibration of masonry material

The first assumption at the base of masonry model is that the thickness of each element representing a block is taken smaller than the wall thickness (200mm). In particular a value equal to the sum of the resistant shells thickness (42mm) of the hollow clay blocks has been adopted. This has be done to arbitrarily take into account the presence of the holes in the masonry blocks. In order to correctly calibrate the model, all the experimental data related to the gross area have been reported to this equivalent thickness.

In order to match the overall stiffness and the compressive behaviour of the masonry, the smeared crack block elements and the mortar interface elements have been consistently calibrated on the basis of the compression tests on the masonry in the direction perpendicular to the holes. This direction has been selected because during the experimental tests the action was applied horizontally to wall and because in this direction the masonry is weaker than in the vertical one. The plasticity and the orthotropic models have been calibrated in order to have the same peak strain (Fig. 4(a)) and a smooth transition between the two models.

Table 1: Parameter adopted in the numerical model.

| SMEARED-CRACK ELEMENTS | | | | | | |
|---|---|---|---|---|---|---|
| **Material** | **E** [MPa] | **$f_c$** [MPa] | **$f'_t$** [MPa] | **$\varepsilon_1$** \ | **$\varepsilon_2$** \ | **t** [mm] |
| *Steel (elastic)* | 210000 | | | | | |
| *Brick* | 20000 | 12.86 | 3 | 0.0018 | 0.00207 | 42 |
| *Wood* | 230 | 6.74 | 1.348 | 0.0073 | 0.00731 | 42 |

| INTERFACE ELEMENTS | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Material** | **$k_{nn}/k_{tt}$** [MPa/mm] | **$s_0$** [MPa] | **$\mu_0$** \ | **$\mu_r$** \ | **$r_0$** [MPa] | **$r_r$** [MPa] | **$G_f^I$** [N/mm] | **$G_f^{II}$** [N/mm] | **$\alpha$** [mm/N] | **$\beta$** [mm/N] | **$\eta$** | **t** [mm] |
| *Brick head joints* | 3364/1488 | 3 | 1 | 0.8 | 0.28 | 0.21 | 0.03 | 0.3 | 0.011 | 12.5 | 0.6 | 42 |
| *Mortar* | 84.18/36.6 | 0.275 | 0.845 | 0.75 | 0.28 | 0.21 | 0.02 | 0.2 | 0.011 | 12.5 | 0.6 | 42 |
| *Mortar on concrete* | 84.18/36.6 | 0.175 | 0.845 | 0.75 | 0.28 | 0.21 | 0.02 | 0.2 | 0.011 | 12.5 | 0.6 | 42 |
| *Sliding joints* | 84.18/36.6 | 0 | 0.42 | 0.42 | 0.02 | 0.02 | 0 | 0 | 0.011 | 12.5 | 0.2 | 42 |
| *Lateral joints on wood* | 84.18/36.6 | 0 | 0.42 | 0.42 | 0.02 | 0.02 | 0 | 0 | 0.011 | 12.5 | 0.2 | 42 |
| *Top joints* | 0.01 | 0.004 | 0.01 | 0.01 | 0 | 0 | 0 | 0 | 0.011 | 12.5 | 0.01 | 42 |

Since fracture tests cannot be easily conducted, the vertical interface element located between the two masonry smeared-crack elements representing the block in tension (Fig. 4(b)), has been calibrated referring to experimental data available in literature [12]. Mode-II fracture energy has been deduced from mode-I fracture energy with the assumption that $G_f^{II} = 10G_f^{I}$.

In order to calibrate the shear behavior of the mortar joints the results of triplet tests have been used. By means of these tests the initial yield surfaces of the mortar interface elements have been estimated (Fig. 4(c)).

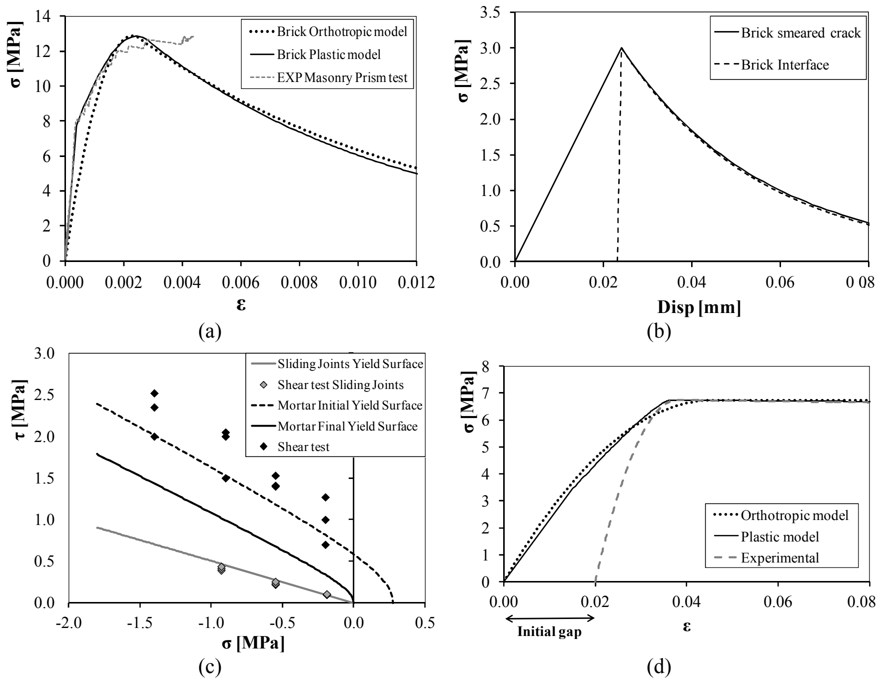


Figure 4: Calibration curves of: smeared crack masonry element in compression (a) and tension (b), interface models for mortar and sliding joints (c) and wood element (d).

## 4.2 Calibration of sliding joints

In the experimental wall the sliding joints were created with wooden boards with a polyethylene sheet. In order to simplify the analysis, in the finite element model they have been modelled using interface elements, calibrated with data obtained from the shear tests presented in Section 3. The model used for these kind of interface elements is the same as the one used for mortar-block interfaces. However, in this case, because of the lack of cohesion due to the presence of the polyethylene sheet between wood and mortar, the hyperbolic surface reduces to the Mohr-Coulomb criterion (Fig. 4(c)), $\mu$ being the coefficient of friction evaluated in the shear tests on triplets.

### 4.3 Calibration of wooden plank properties transverse to the fibers

In the numerical models, the two wooden vertical elements introduced between the column and the infill have been modelled with smeared crack elements. The material has been calibrated in order to reproduce the experimental compressive behaviour of wood transverse to the fibers, considering the effective contact surface evaluated after the test on the wall. The graph in Fig. 4(d) shows the calibration of the plasticity and orthotropic models. The elastic stiffness has been taken lower than the experimental value. Such discrepancy is justified by the need to take into account the presence of an initial partial detachment (gap, Fig. 4(d)) between the boards and the vertical columns of the frame, due to the imperfect planar surface of the wooden boards, caused by the natural drying process.

### 4.4 Comparison of numerical and experimental infilled frame response

The reliability of the numerical model has been verified comparing the FEAP model results with the force-drift curves obtained from in-plane tests on masonry infills with and without horizontal sliding joints. Fig. 5 underlines a good correspondence between the results. The model also reproduces quite well the failure pattern of the tests. In particular for the solid infill the damage is governed by stair-stepped diagonal cracks along the mortar joints diagonally oriented in the wall. Only some additional crushing of the masonry units are obtained through the finite element model, not observed in the test.

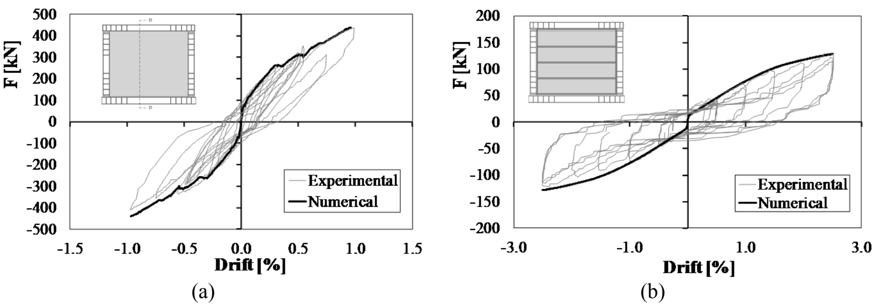


Figure 5: Comparison between numerical and experimental results for the solid infill (a) and for infill with sliding joints (b).

For the infill with sliding joints the in plane behaviour is characterized by the sliding along the horizontal wooden boards and by the plastic compressive strain of the lateral wooden boards at the masonry subportion corners. No damage occurs in the masonry, in accordance with the test results. The model clearly highlights the role played by the lateral wooden boards that has been observed experimentally during the in plane test. As already discussed, the vertical boards act like soft cushions, being the elastic modulus of wood far lower compared to the one of masonry and its compressive strength smaller than the masonry one. At the end of the analysis, as at the end of the experimental test, there is no damage in the wall, only cracking at the interface between masonry and wood due to the sliding along the horizontal sliding joints.

# 5 Parametric analysis

Moving from numerical models above discussed, a parametrical analysis has been performed on some design quantities to understand their role on the nonlinear response of masonry infills, realized with the proposed technique. The role of the parameters relating to the mechanical proprieties of the component materials has been investigated, i.e. the elastic modulus and the compressive strength of masonry and wood (Fig. 6(a) and (b)). Furthermore the role of parameters related to the configuration of the sliding joints, such as the variation in the number of joints (Fig. 6(c)) and in the coefficient of friction along the joints themselves (Fig. 6(d)) has been studied.

The role of changes related to the geometry of the infill has also been taken into account, varying the length (Fig. 6(e)), the thickness (Fig. 6(d)) and the role played by the top gap between the wall and the top beam of the frame.

In the graph in Fig. 6(a), it can be seen that the lateral response of the infill changes dramatically; both increasing the compressive strength of the vertical wooden board, maintaining constant the mechanical properties of masonry, or reducing the strength of masonry, keeping constant those of wood. In either case, the crushing of the masonry subportions corners occurs due to a ratio of wood over masonry strength larger than unity. In the figure, the damage in the masonry is testified by an abrupt change in the stiffness of the response. Also initial stiffness of the infill varies with a more significant dependence on the wood stiffness.

These results highlight the importance of the hierarchy of strength approach in the choice of materials used for the construction of the infill. The yielding compressive stress of the smeared crack element between the infill and the column has to be lower than masonry compressive strength, in this way the infill can be protected by local crushing and by the consequent arising of extensive damage. The fundamental role played by the lateral wooden boards, for the "good" behaviour of the infill, is supported by the analysis results shown in Fig. 6(b). In these models the lateral wooden boards have been removed, while maintaining the horizontal sliding joints and varying the elastic modulus of masonry, in the range from 50MPa (lower limit as for adobe masonry) to 4500MPa (tested masonry), with a constant strength of the materials. The results show that, decreasing the stiffness of the masonry, the system response changes, with the damage that reduces decreasing the masonry stiffness. This is because the reduction of the elastic modulus of masonry involves an increase of the deformation capacity of the masonry subportions and therefore increases its ability to accommodate large deformations without failure. The results show that, for range of stiffness $E_m$=50÷250MPa (typical of earthen masonry), the lateral wooden boards are not required to obtain an in-plane behaviour without damage [6]. On the contrary, for stiffer masonry, a protection of the subportion corners has to be supplied.

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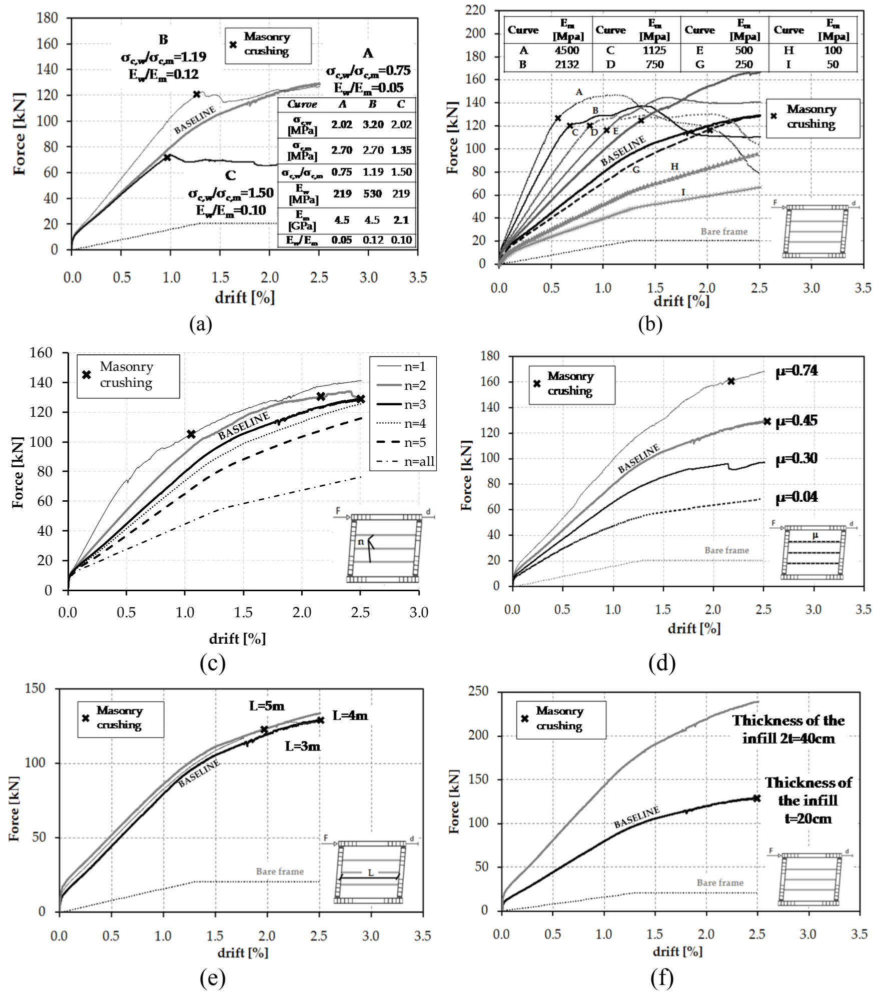



Figure 6: Parametric analyses results varying the mechanical properties of masonry and wood (a, b), varying the sliding joints number (c) and friction coefficient on vertical and horizontal masonry-wood interfaces (d) and varying the geometry of the infill as length (e) and thickness (f).

In Fig. 6(c) is shown the response of the system obtained by varying the number of horizontal sliding joints, within the range of zero (no sliding joints) to eleven (one wooden board in each mortar bed joint). Results show that increase in number of sliding joints corresponds to a reduction in the strength and stiffness of the system response.

The variation of the friction coefficient between wood and masonry has been investigated in the case of sliding joints and vertical wooden boards on the columns made with different materials. Fig. 6(d) shows that, increasing the coefficient of friction, the lateral strength and stiffness of the infill are increased.

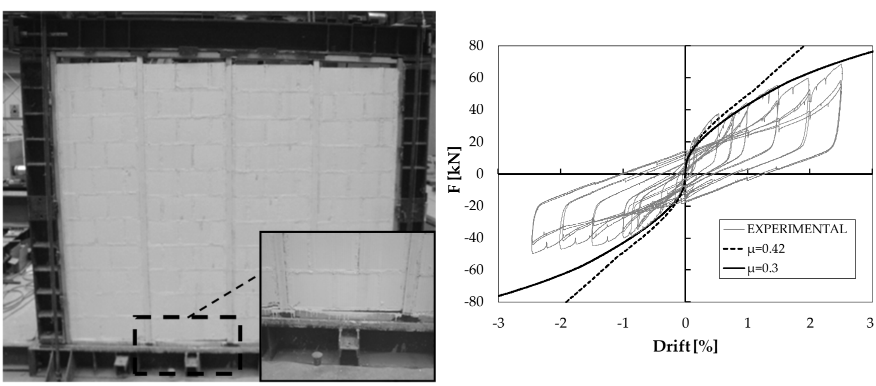
Fig. 6(e) shows the results obtained by modelling the infill with variable length, starting from a minimum of 3 m (corresponding to the experimental wall) to 5 m. The resistance of the infill grows slightly increasing the length. This increment is probably due the higher weight of the wall which increase the normal stress on the sliding joints and their friction resistance. As a consequence, higher lateral loads are required to activate the sliding, as it is visible at very low drift levels. In general, however, the lateral response of the infill does not change significantly with the increase of the length, the predominant deformation mechanism remains the sliding along the horizontal joints with compressions located at the sub-portions corners.

The second geometrical parameter investigated is the thickness of the wall (Fig. 6(f)). A thickness twice than the one of the reference model is considered. The result shows that the response of the infill is proportional to the thickness. No changes in the deformation mechanism is evidenced.

The results show that the main contribution to the infill resistance is given by the activation of diagonal struts inside each masonry subportion that characterizes the contact stress between the masonry and the frame columns.

To extend the parametric analysis on the configuration of sliding joints, in Fig. 7(a) the preliminary results on the in-plane response of a masonry infill wall with vertical sliding joints, instead of horizontal, are presented. Both experimental and numerical results are presented. The mechanism of deformation observed is characterized by the rotation of the wall partitions around their base corners together with their relative sliding along the vertical boards. The measured in-plane resistance was lower than in case of horizontal joints, and no damage in the masonry was observed up to a drift of 2.5%.

The numerical modelling, previously calibrated, was used also to describe the response in the case of vertical joints configuration. The results are shown in




(a) (b)

Figure 7: View of the infill with vertical wooden joints at 2.5% drift (a) and comparison of the Force-Drift experimental and numerical curves (b).

Fig. 7(b). It is worth noting that the model has an inherent difficult in reproducing the friction phenomena along the vertical sliding joints, due to the imperfect contact condition between the mortar and wooden boards. In fact the gaps at these interfaces that close during the wall deformation, influence the normal stress in the sliding joints, and the consequent friction. In Fig. 7(b) the influence of the gap is highlighted by plotting the numerical response obtained adopting the experimental coefficient of friction (0.42) and a reduced value (0.3), which tries to take into account a reduced normal compression along the sliding joints due to the presence of an initial gap on each wooden plank-masonry interface.

## 6 Conclusions

The paper describes the results of a parametric analysis on the response of infill walls with sliding joints. The role of mechanical and geometrical parameters of the components has been investigated, in order to define some design details for the best behaviour of the infill. The results highlight the role of masonry mechanical properties in the control of damage in the infill. For very low stiffness, as typical of earthen masonry, for example, sliding joints can nullify the damage in the infill under in-plane loading. In presence of stiffer masonry material, crushing at the masonry subportion corners can be avoided only thanks to the introduction of a weaker material between the columns and the infill (namely a wooden board) in order to absorb the local required deformations. Increasing the number of horizontal sliding joints partitioning the wall, the infill in-plane resistance is reduced. A similar reduction of infill strength can be obtained by reducing the friction between the masonry and the partitioning elements. An even more effective reduction of strength (about -50%) has been obtained in a tested wall with the sliding joints placed in the vertical position. In this case the preliminary results show that the mechanism of deformation is governed by the rocking mechanism of the masonry subportion on their base, without significant damage in the masonry, even for very large deformations applied (2.5% drift).

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## References

[1] Calvi M., Bolognini D. “Seismic response of reinforced concrete frames infilled with weakly reinforced masonry panels”. *Journal of Earthquake Engineering*, 2001, 5:2, 153-185.

[2] Sigmund V, Penava D (2014) Influence of openings, with and without confinement, on cyclic response of infilled RC frames: an experimental study. J Earthq Eng **18**(1): pp. 113-146, doi:10.1080/13632469.2013.
[3] Ezzatfar, P., Binici, B, Kurc, O., Canbay, E., Sucuoglu, H. and Ozcebe, G. (2014) "Application of mesh reinforced mortar for performance enhancement of hollow clay tile infill walls," *Seismic Evaluation and Rehabilitation of Structures*. Springer, Volume 26, pp. 171-186.
[4] Mohammadi, M. & Akrami, V. An engineered infilled frame: Behavior and calibration. *Journal of Constructional Steel Research* 2010, **66**(6), pp. 842-849.
[5] Preti M., Bettini N., Plizzari G. Infill walls with sliding joints to limit infill-frame seismic interaction: large scale experimental test. *Journal of Earthquake Engineering* (2012).
[6] Preti M, Migliorati L, Giuriani E (2014) Experimental testing of engineered masonry infill walls for post-earthquake structural damage control, *Bulletin of Earthquake Engineering*, published on-line, DOI: 10.1007/s10518-014-9701-2.
[7] Migliorati L. "Studio delle prestazioni sismiche e dei criteri progettuali di tamponamenti duttili con giunti di scorrimento". *PhD Thesis*, DICATAM, University of Brescia, Italy, 2014.
[8] Stavridis A, Shing P.B (2010), Finite-Element Modelling of Nonlinear Behavior of Masonry-Infilled RC Frames, *Journal of Structural Engineering* **136**(3): pp. 285-296.
[9] Lofti H., Shing B.P. Interface model applied to fracture of masonry structures. *Journal of Structural Engineering* 1994, **120**(1), pp. 63-80.
[10] Van der Pluijm (1993), Shear behavior of bed joints, Proceedings of the 6th North American Masonry Conference, Boulder, Colorado.