SHEAR BEHAVIOUR OF FRP RETROFITTED MASONRY WITH UNCERTAINTIES IN THE MATERIAL PROPERTIES: PARAMETRIC STUDY

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Abstract. *Masonry structures shear failure is generally preceded by a massive cracking development in the mortar joints. For this reason, the mortar joints limit the final strength. Mechanical properties degradation and structural safety loss make the rehabilitation or reinforcement necessary. The reinforcement technique with fiber reinforcement polymers has experimentally proved to be very effective. However, in the present, analytical and numerical capacity to quantify this retrofitting system efficiency is still subject of research. In order to improve these intervention techniques it is necessary to have more experimental data and a numerical tool to simulate their behavior. Although the in-plane mechanical behaviour of unreinforced masonry and masonry retrofitted with fiber reinforced polymer (FRP) laminates has been studied by several authors, the analytical modelling of mechanical behaviour of masonry remains an open problem, due to its natural variability and inhomogeneity. A coupled damaged-plasticity model in which bricks and mortar are separately modelled (micro approach) is considered to carried out a parametric study of in plane shear behavior of unreinforced and FRP retrofitted scaled masonry walls. Composite materials are simulated as an elastic orthotropic material. Parametric study results are compared with experimental data obtained from unreinforced and FRP reinforced panels subjected to shear loads.*

Keywords: *Masonry, Fiber Reinforced Polymers, Retrofitting, Shear strength*

1 INTRODUCTION

Masonry is a non-homogeneous composite anisotropic material made of hollow or solid bricks. The shear behaviour of masonry units is complex even under very low load levels. A high number of factors affect its behaviour, among which are the mechanical and geometrical properties of units and mortar, the characteristics of their interfaces, the geometrical arrangement of masonry and the quality of workmanship.

Masonry structures shear failure is generally preceded by a massive cracking development in the mortar joints. For this reason, the mortar joints limit the final strength. Depending on compression degree, failure can take place only in the joints, or a combined crack brickmortar joint can occur. Failure is normally brittle and sudden. One way of preventing this type of problem is through FRP reinforcement. This reinforcement can be done covering the entire wall with fabrics of carbon or glass impregnated in epoxy resin or through strips applied in the same way.

The reinforcement and rehabilitation technique with fibre reinforced polymers (FRPs) has experimentally proved to be very effective. If the reinforcement is properly designed this technique may enhance the in-plane shear behaviour, increasing ductility and in some cases, ultimate strength and stiffness (ElGawady, 2004; Valluzzi et al., 2002; Rougier, 2007; Luccioni and Rougier, 2011).

In order to analyze the influence of different mechanical properties of constituent materials of masonry, a parametric study on unreinforced and carbon fiber reinforced polymers (CFRP) retrofitted masonry panels subjected to diagonal compression was performed. This is part of an ongoing research focusing on the study of the stochastic model. The following parameters were considered: Youngs modulus (E) of the clay bricks and mortar, Youngs modulus of CFRP and number of layers of CFRP laminate.

To carry out that study an existing coupled damaged-plasticity model (Luccioni and Rougier, 2005) was used. Such model allows simulating the behaviour of masonry elements using the mechanical properties of constitutive materials and their layout and it also permits to take into account the debonding of the interface between bricks and mortar.

First, a summary of the main characteristics of the mechanical behaviour of unreinforced and retrofitted masonry, subjected to shear loads is presented. After that, the model used in this paper is described and the main coupled damaged-plasticity orthotropic model equations used are developed (Luccioni et al., 1995, 1996; Luccioni and Rougier, 2005). Finally, some load-displacement curves are shown and numerical and experimental results are compared.

2 SHEAR BEHAVIOUR OF UNREINFORCED CLAY MASONRY UNITS

Masonry walls are primarily designed to resist axial loads. However, they are often subjected to in-plane or out- of- plane forces resulting from lateral loads such as earthquakes. The in-plane shear resistance of load bearing unreinforced masonry (URM) walls is provided by the shear bond strength of the mortar and the friction shear due to the vertical load. Under severe earthquakes loads the shear capacity of the mortar is exceeded, resulting in failure of the wall (Ehsani et al., 1997).

The typical shear failure modes shown by masonry walls subjected to in-plane forces are: sliding along the bed joints, diagonal tension cracking and shear compression failure (Fig. 1).

The first two modes are the most common. Sliding along the bed joints is a failure in which fissure appears along the horizontal joints and it is produced when the units strength is higher compared to the mortar bond strength. Consequently, cracking occurs on the weakest element, which in this case is the joint. This causes the sliding of the upper part of the wall onto the lower part (Fig. 1a).

The second failure mode is generally characterized by a diagonal tension failure. This type of failure normally occurs when tension strength units is low compared with bond strength mortar. In general, when there are no compression forces or when they are very small, the failure tends to occur following bed and head joints (staggered cracks) (Fig. 1b). When compression is applied cracks can cross over bricks and the failure angle becomes dependent on its magnitude. (Gallegos, 1993). The probability that the cracks spread through the units increases with the vertical compression load (ElGawady, 2004).

Shear compression failure is characterized by the cracking of compressed zones at the wall sides and it makes the wall overturn (Fig. 1c).

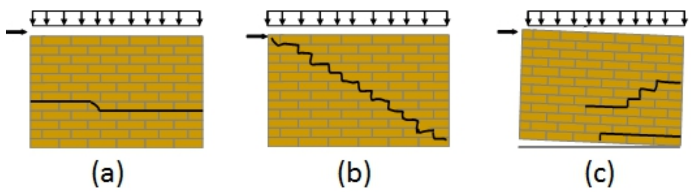


Figure 1: Modes of failure in URM walls subjected to in-plane forces: (a) Sliding along the bed joint. (b) Diagonal tension cracking. (c) Shear compression failure.

3 SHEAR BEHAVIOUR OF FRP REINFORCED CLAY MASONRY UNITS

Some researchers have been focused on enhancing the in plane shear capacity of URM walls by means of external reinforcement of composite materials (CFRP and GFRP. Under shear loads and in absence of normal stresses, failure of unreinforced clay masonry panels generally occurs due to mortar joints sliding which causes a very brittle and sudden failure. Depending on the dimensions and configuration chosen, FRP reinforcement modifies this brittle behaviour, avoiding the sliding of the joints and increasing ultimate strength, stiffness and, in some cases, ductility (Valluzzi et al., 2002; ElGawady, 2004; Gabor et al., 2006; Rougier, 2007; Santa María et al., 2006; Papanicolaou et al., 2011). The URM retrofitting with FRPs is one method that attempts to improve a structures load carrying capacity and integrity during an earthquake event (Zhuge, 2010).

FRP reinforcement is generally made covering the entire wall with fabrics of carbon or glass impregnated in epoxy resin and then applied over the surface previously primed, or through strips applied in the same way (wet process) (Valluzzi et al., 2002; Chuang et al., 2003; ElGawady, 2004; ElGawady et al., 2005; Gabor et al., 2006; Alcaino and Santa María, 2008; Rougier, 2007; Prakash and Alagusundaramoorthy, 2008; Santa María and Alcaino, 2011). Some FRP in plane retrofitting schemes for masonry walls are presented in Fig. 2.

A quite remarkable gain in strength, an important deformation capability and negligible increasing in-plane stiffness was obtained by Gabor et al. (2006) on hollow brick masonry panels entirely reinforced with bidirectional glass fiber composite. Under dynamic tests, in general,

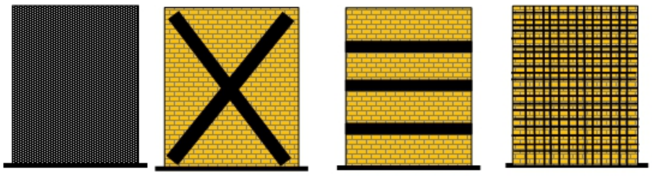


Figure 2: URM in plane FRP retrofitting schemes (a). Composite sheets bonded to entire wall surface. (b) Cross arrangements of FRP plates or laminates. (c) Horizontal strips of FRP laminates. (d) grid disposition of FRP.

bi-directional surface type materials (fabrics and grids) bonded to the entire wall surface and correctly anchored can help to postpone the three classic failure modes of masonry walls: rocking ("flexural failure"), step cracking and sliding ("shear failures"). Additionally, in some situations, this retrofitting scheme can postpone in-plane collapse by "keeping the bricks together" under large seismic deformations (ElGawady, 2004).

Unidirectional strips reinforcement arranged along diagonal direction, improves shear strength and preserves specimen integrity after cracking. The length of FRP bands' anchorage constitutes a very important design variable. The longer it is, the higher the ultimate strength reached by the masonry element will be (Valluzzi et al., 2002; Gabor et al., 2006; Rougier, 2007).

4 NUMERICAL STUDY

In this work the masonry and CFRP reinforced masonry elements under diagonal compression are simulated with a micro-model in which bricks and mortar are separately modeled (Luccioni and Rougier, 2011). The model was implemented in a non-linear plane finite element program.

The application of this model to masonry requires the proper definition of the finite element model and the definition of several functions (yield function, potential function, damage function, hardening functions, etc.) and parameters for the constitutive models. Functions normally used for concrete were used for bricks and mortar. Some of the mechanical properties of bricks and mortar, like elasticity modulus, Poison ratio, compression strength, tension strength, fracture energy and compression strain hardening were obtained from tests performed on bricks and mortar specimens (Rougier, 2007). The rest of the parameters were indirectly obtained through numerical simulation of small unreinforced masonry specimens and comparison with experimental results (Luccioni and Rougier, 2011).

The interface between bricks and mortar allowing possible debonding is also approximately considered without explicitly modeling the interface but properly modifying the mortar constitutive equation (Luccioni and Rougier, 2011; Luccioni et al., 2005). The mortar was supposed to be anisotropic with reduced shear strength in the interface plane. The value of the shear strength was adjusted to reproduce the experimental results of small masonry specimens tested in (Luccioni and Rougier, 2010). FRP reinforcement is simulated without explicitly modeling reinforcement elements but with a generalization of the classic mixture theory (Oller et al., 2006). In this way, FRP reinforced bricks or FRP reinforced mortar are considered as composite materials made of bricks or mortar and FRP composite respectively. The mixture theory is applied to these composites and the modified mixture theory is used for the FRP composite itself (Luccioni, 2006; Toledo et al., 2008).

4.1 Constitutive model for solid clay bricks, mortar

Orthotropic plastic-damage models (Betten, 1988; Luccioni et al., 1995; Luccioni and Rougier, 2005; Luccioni et al., 1996) are used for bricks and mortar in the micro-model. Even though different materials with different mechanical properties are dealt with, solid clay bricks, mortar and masonry are frictional materials, that is, their behaviour is influenced by hydrostatic pressure. Bricks are normally made of isotropic materials and mortar is also approximately isotropic. However, mortar and bricks are modelled as orthotropic materials in order to account for the weakness of the interfaces and the possibility of relative displacements without explicitly modelling the interfaces (Luccioni et al., 2005).

4.2 Anisotropy treatment

The anisotropic model is based on the assumption that two spaces can be defined (Betten, 1988; Luccioni et al., 1995): (a) a real anisotropic space and (b) a fictitious isotropic space. The problem is solved in the fictitious isotropic space allowing the use of elastoplastic models originally developed for isotropic materials (Luccioni and Rougier, 2013).

4.3 Isotropic model: general form

The plastic process is described by a generalization of classical plasticity theory that takes into account many aspects of geomaterials behavior (Luccioni et al., 1996). The elastic threshold is described by a yield function,

$$F(\sigma_{ij}, \alpha_k) = f^p(\sigma_{ij}) - K(\sigma_{ij}, \kappa^p) \leq 0 \tag{1}$$

where $f^p(\sigma_{ij}$ is the equivalent stress defined in the tension space and that can take up the form of any of the yielding functions of classic plasticity (Tresca, Von Mises, Mohr Coulomb, Drucker Prager, etc). If this model is used for mortars or bricks a suitable approach for frictional materials as Mohr Coulomb or Drucker Prager must be adopted. $K(\sigma_{ij}, \kappa^p)$ is the yielding threshold and κ^p is the plastic hardening variable.

The following rules are used for the evolution of plastic strains:

$$\epsilon_{ij}^p = \lambda \frac{\partial G(\sigma_{mn}, \kappa^p)}{\partial \sigma_{ij}} \tag{2}$$

where λ is the plastic consistency factor and G is the plastic potential function.

The plastic hardening variable κ^p is obtained normalizing energy dissipated by the plastic process to unity and varies from 0, for the virgin material, to 1 when the maximum energy is plastically dissipated (Luccioni and Rougier, 2005; Luccioni et al., 1996; Rougier and Luccioni, 2007).

The damage threshold is described by a damage function in the following way:

$$F^d = f^d(\sigma_{ij}) - K^d(\sigma_{ij}, \kappa^d) \leq 0 \tag{3}$$

where $f^d(\sigma_{ij})$ is the equivalent tension, $K^d(\sigma_{ij}, \kappa^d)$ is the equivalent damage threshold and is the hardening variable (Luccioni et al. 1996). The equivalent tension $f^d(\sigma_{ij})$ may be evaluated using known yielding functions (Tresca, Von Mises, MohrCoulomb or DruckerPrager) or any function specially developed for damage (Luccioni et al., 1996; Luccioni and Rougier, 2005).

The scalar damage variable d varies from 0 to d_c. that is $0 \leq d \leq d_c$ where $0 \leq d_c \leq 1$ is the level of damage correspondent to the material failure.

The evolution of permanent strains and damage is obtained from the simultaneous solution of the following equations called the consistency conditions of the problem (Luccioni et al., 1996),

$$\begin{cases} \dot{F}^p = 0 \\ \dot{F}^d = 0 \end{cases} \tag{4}$$

Equations 4 are two linear equations in $\dot{\lambda}$ and $\dot{d}$

4.4 Composite materials modeling

The reinforcement material made up of polymeric matrix and carbon fibres is itself a composite material formed by a matrix with embedded long fibres. To simplify the numerical simulation and to reduce calculus volume, it was modelled with an equivalent homogeneous model. An orthotropic elastoplastic model with the composite properties was used for that purpose. As the properties provided by the manufacture were not enough to model this material, a generalization of mixture theory (Luccioni, 2006; Toledo et al., 2008) was used to obtain all the reminding mechanical properties. In this calculus, the properties of the composite were obtained from the properties of the fibres and the epoxy matrix and the fibres volume ratio. In this way, the lamina properties already given by the manufacture were also verified (Rougier, 2007).

4.5 CFRP reinforced masonry

The CFRP laminas applied on both faces of the masonry panels were not modelled independently but together with brick or mortar. This consideration gives place to a composite material consisting of brick or mortar and two sheets of composite. In all cases the finite element mesh should be carefully defined so that the elements match the reinforcement zones

Mixtures theory can be used to model the in plane behaviour of this type of composite where the strains are the same for both component materials (Rougier, 2007).

5 PARAMETRIC STUDY

A parametric study is presented in this section. This is based on an existing finite element modeling (Luccioni and Rougier, 2005) and it was performed on unreinforced panels and panels entirely reinforced with unidirectional CFRP laminates subjected to shear loads. The dimensions of the panels were 580 mm $\times$ 610 mm $\times$ 130 mm and they had 15 mm-thick mortar joints.

In order to analyse the effects on the load carrying capacity and stiffness of unreinforced and retrofitted masonry, the influence of different values of some mechanical properties of bricks, mortar and CFRP were studied.

First, the effect of slight variation of Young's modulus of the units and mortar on the ultimate load of unreinforced masonry was analyzed. Then, Young's modulus of CFRP was varied, while the mechanical properties of bricks and mortar remained constant. Finally, the effect of different number of CFRP layers on ultimate shear strength and stiffness of retrofitted masonry was studied. The aim was to understand the global behaviour of CFRP retrofitted masonry under shear loading.

5.1 Influence of the Young's modulus and compressive strength of the clay bricks and mortar on load carrying capacity and stiffness of unreinforced masonry panels

The results of a numerical study carried out in order to know the influence of the Young's modulus of the units and mortar in masonry final shear strength and stiffness are shown in this section. Diagonal compression tests were simulated with a non-linear plane finite element program in which the models described in sections 4.1 to 4.4 were implemented. The results of these simulations are hereafter presented.

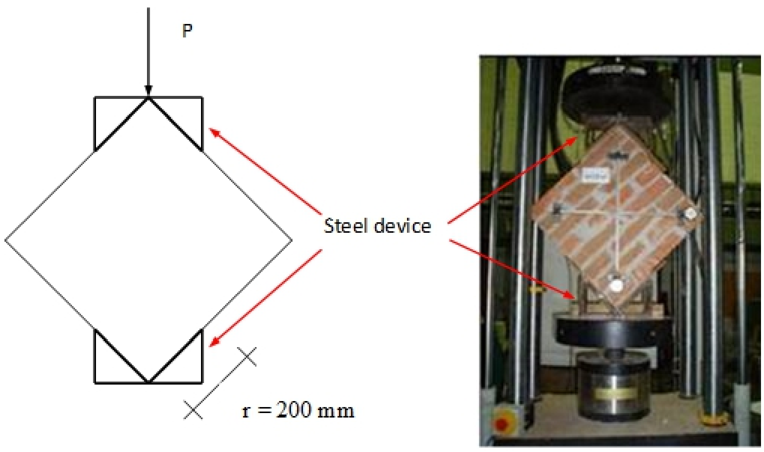


Figure 3: Diagonal compression test setup and measurement devices arrangement.

The recommendations of INPRES CIRSOC 103 propose an inclined compressive loading test on masonry panels in order to estimate the masonry shear strength. According to these prescriptions (INPRES CIRSOC 103, 1991), test panels should be square and with more than 550 mm side. In order to apply the compression loading, the corners of the panels should be embedded in two metallic supports and with embedding length r greater or equal to 200 mm. The test setup and measurement devices arrangement used for diagonal compression tests are shown in Fig. 3

The whole panel was modelled and three node triangular plane stress finite elements were used. Masonry elements were meshed distinguishing bricks and mortar elements. The mechanical properties of mortar and bricks were experimentally obtained and are summarized in Table 1 (Rougier, 2007).

The loading conditions and finite element (FE) mesh for masonry panels tested under diagonal compression are presented in Fig. 4.

Table 1: Bricks and mortar mechanical properties corresponding to 580 mm × 610 mm × 130 mm panel

Properties	Mortar	Clay Brick
Elasticity Modulus, E (MPa)	1904	1662
Poissons ratio, ν	0.21	0.15
Tension ultimate strength, σ_{ut} (MPa)	0.38	0.35
Compression ultimate strength, σ_{uc} (MPa)	4	7.20
Uniaxial compression elastic threshold, σ_{fc} (MPa)	3.6	—
Initial compression/tension strength ratio, R_0^p	10.5	20.5
Fracture energy, Gfp (MPa mm)	1.2E-03	2.0E-03
Crushing energy, Gc p (MPa mm)	1.2	1.4
Yield criterion	Lubliner-Oller	Lubliner-Oller
Plastic flow	Lubliner-Oller	Lubliner-Oller

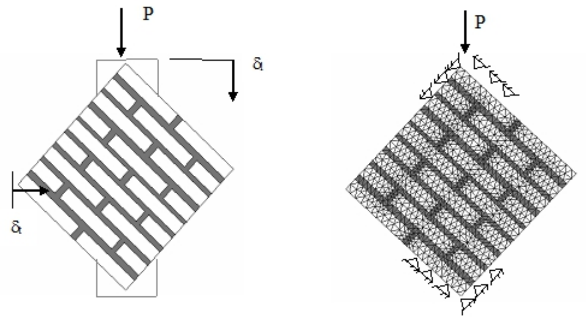


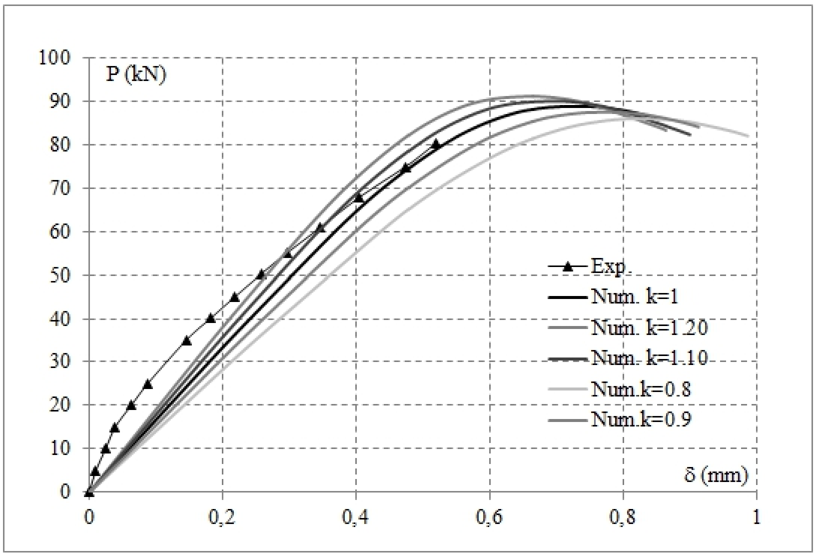
Figure 4: . Unreinforced panel under diagonal compression: (a) loading conditions; (b) FE mesh.

Numerical loaddisplacement curves (P-δ), through the compressed diagonal of unreinforced masonry panels and their comparison with experimental results are presented in Figs. 5 and 6.

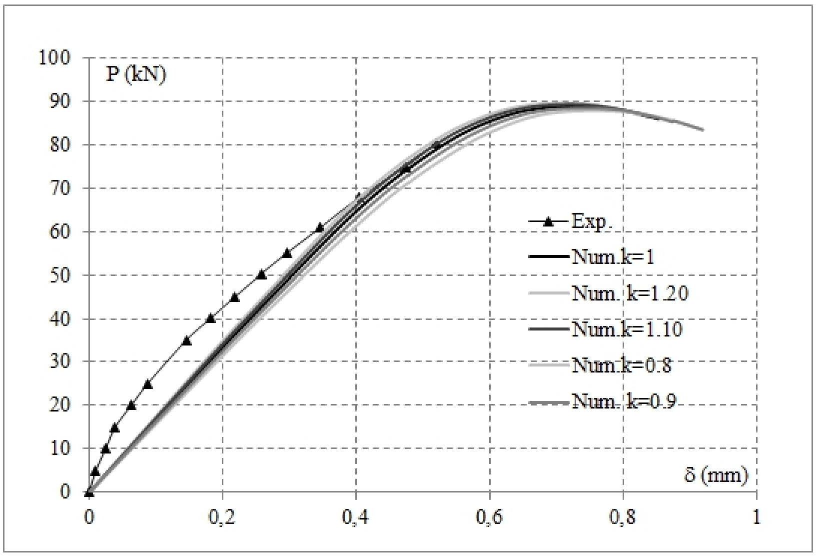
Relative displacements along the compressed diagonal were recorded between two fixed points located along the diagonal and were later extrapolated to the total length of the diagonal to obtain the total diagonal displacements. Variations of a proportional factor k of the Youngs modulus and ultimate compressive strength of bricks are shown in Figs. 5a and 5b, respectively. Then, the same variations of Young's modulus and compressive strength were considered for the mortar and the load-displacement diagrams are presented in Figs. 6a and 6b, respectively.

Finally, variations of a proportional factor k of the shear strength of the mortar in the interface plane are presented in Fig. 7. In this case the mortar was supposed to be anisotropic in order to account for the weakness of the interfaces and the possibility of relative displacements without explicitly modelling the interfaces (Luccioni et al., 2005).

It can be seen that maximum load and stiffness of masonry under uniaxial shear loading depends chiefly on Youngs modulus of the bricks (see Fig. 5a). On the other hand, the ultimate



(a)



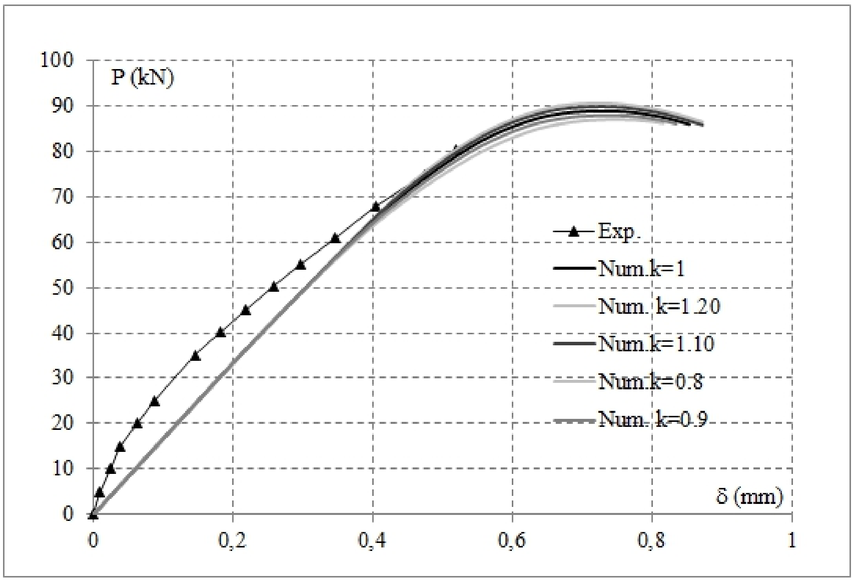
(b)

Figure 5: Unreinforced panel under diagonal compression: (a) loading conditions; (b) FE mesh.

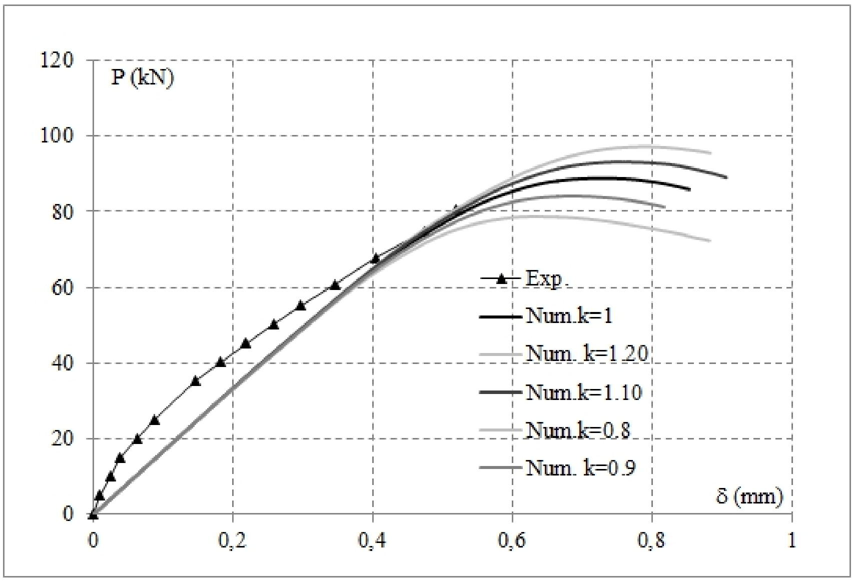
compressive strength of the brick units has practically no influence on the ultimate load capacity and stiffness of masonry (Fig. 5b).

Variations of the Young's modulus of mortar produce small variations of load carrying capacity and stiffness of masonry, as can be observed in Fig. 6a. However, variations of the ultimate compressive strength of the mortar modified significantively the ultimate load capacity of masonry while stiffness of masonry remains practically unmodified (Fig. 6b).

As it can be seen in Fig. 7, increase of the shear strength of the mortar implies a considerable increase of the ultimate load capacity of masonry. The load capacity of masonry increases about 20 % for a k factor equal 0.8 in comparison with $k = 1$. When k factor is less than 1 the anisotropy of mortar is reduced, shear strength increases and in consequence the bond between bricks and mortar is stronger with minor possibility of relative displacements.



(a)



(b)

Figure 6: Unreinforced panel under diagonal compression loading: a) Variation of the Young's modulus of the mortar; b) Variation of the ultimate compressive strength of the mortar

Numerical values of maximum load of masonry corresponding to different values of proportional factor (k) applied to Young's modulus (E) and ultimate compressive strength of bricks and mortar are shown in Table 2 and Table 3, respectively.

The influence of the Young's modulus of the bricks in the specimen's response to diagonal compression loading is also clear in Table 2. The maximum difference of maximum load of masonry is about 2.7 % and it corresponds to an increase of the Young's modulus of the bricks of 1.2 times. There is no significantively improvement in the shear strength resistance, when Young'ss modulus of the bricks is increased. The maximum difference of stiffness of masonry is about 14 % and it corresponds to an increase of the Youngs modulus of the bricks of 1.2 times.

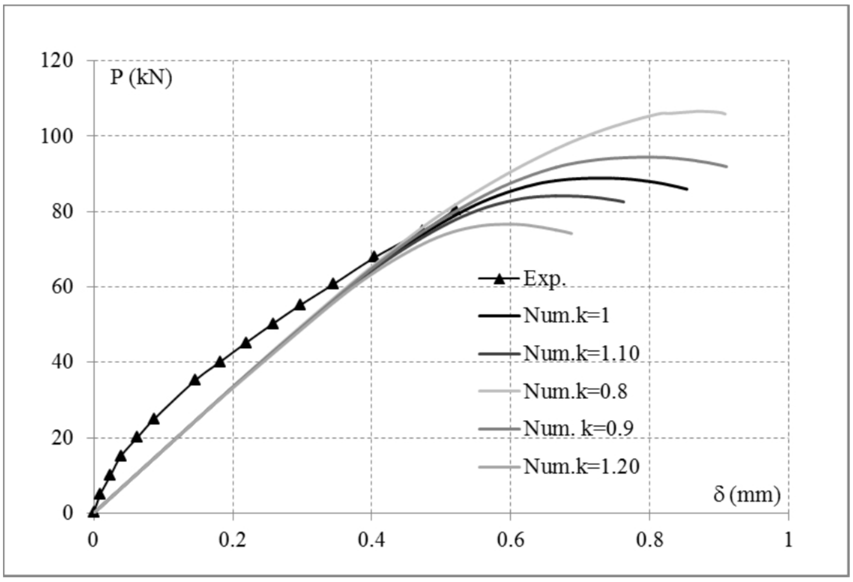


Figure 7: Unreinforced panel under diagonal compression loading: Variation of the shear strength of the mortar

Table 2: Maximum load and stiffnes of unreinforced masonry for different values of k factor applied to Young's modulus (E) of bricks and mortar

k factor	P_{max} (kN)		Stiffness (kN/mm)	
	Brick	Mortar	Brick	Mortar
0.8	86.03	87.87	141.43	157.52
0.9	87.53	88.44	154.52	162.52
1.00	88.89	88.89	166.85	166.85
1.10	90.09	89.65	178.76	170.76
1.20	91.22	89.34	190.14	174.23

Table 3: Maximum load and stiffnes of unreinforced masonry for different values of k factor applied to ultimate compressive strength (σ_c) of bricks and mortar

k factor	P_{max} (kN)	
	Brick	Mortar
0.8	87.01	78.80
0.9	87.91	84.21
1.00	88.89	88.89
1.10	89.86	93.24
1.20	90.61	97.25

In that case, the higher Young modulus of the bricks the higher is the stiffness of masonry panel.

5.2 Influence of the Young's modulus and number of layers of CFRP reinforcement on load carrying capacity and stiffness of retrofitted masonry

Panels entirely retrofitted with CFRP laminates and subjected to diagonal compression loading were numerically analyzed. The study variables were Young's modulus and number of layers of the CFRP reinforcement.

Table 4: Composite mechanical properties.

Volume fraction of fibres, k_f	0.3
Longitudinal elasticity modulus, E_l (MPa)	72500
Transversal elasticity modulus, E_t (MPa)	6200
Longitudinal-transversal Poisson's ratio , ν_{lt}	0.08
Transversal-longitudinal Poisson's ratio , ν_{tl}	0.017
Transversal-transversal Poisson's ratio, ν_{tt}	0.20
Longitudinal tensile strength,σ_{ul} (MPa)	960
Transverse tensile strength,σ_{ut} (MPa)	51
Yield criterion	Tresca

Unidirectional carbon fibre fabric with high content of carbon and high modulus and strength, saturated in situ with an epoxy system was used for the reinforcement. The properties of the lamina were obtained from the manufacturer (SIKA) specifications and were numerically validated (Rougier, 2007) using a constitutive model for unidirectional fibre reinforced lamina. They are presented in Table 4.

The whole panel was modelled. Triangular plane stress elements with three nodes were used for the simulations. The evolution of longitudinal displacements (δ_l) as a function of the applied load corresponding to different values of proportional factor (k) applied to Young's modulus (E) of CFRP laminate is presented in Fig. 8. No appreciable increase in ultimate load is observed while stiffness is kept practically unmodified.

Loads versus longitudinal displacements curves obtained for different number of CFRP layers are presented in Fig. 9. In this case, a significant increase in ultimate shear load, ductility and stiffness can be observed when the number of CFRP layers is increased. However, due to high costs of CFRP laminates, the increase of the number of layers of these materials is expensive.

Numerical values of maximum load of masonry corresponding to different values of proportional factor applied to Young's modulus of CFRP laminates are shown in Table 5.

The maximum difference of maximum load of retrofitted masonry is about 5.4% and it corresponds to an increase of the Young's modulus of the CFRP laminates of 1.2 times. When

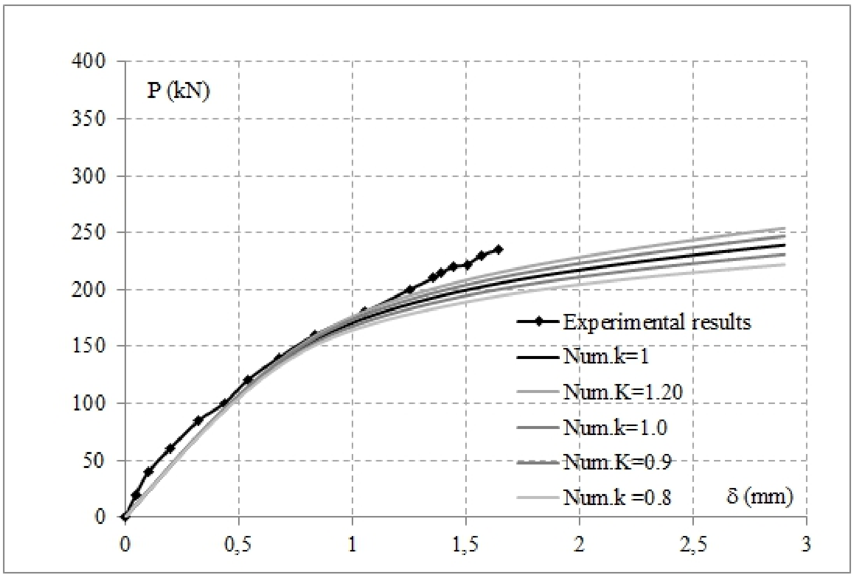


Figure 8: Load-displacement diagram of the compressed diagonal of CFRP reinforced panel: Variation of the Young's modulus of CFRP laminates

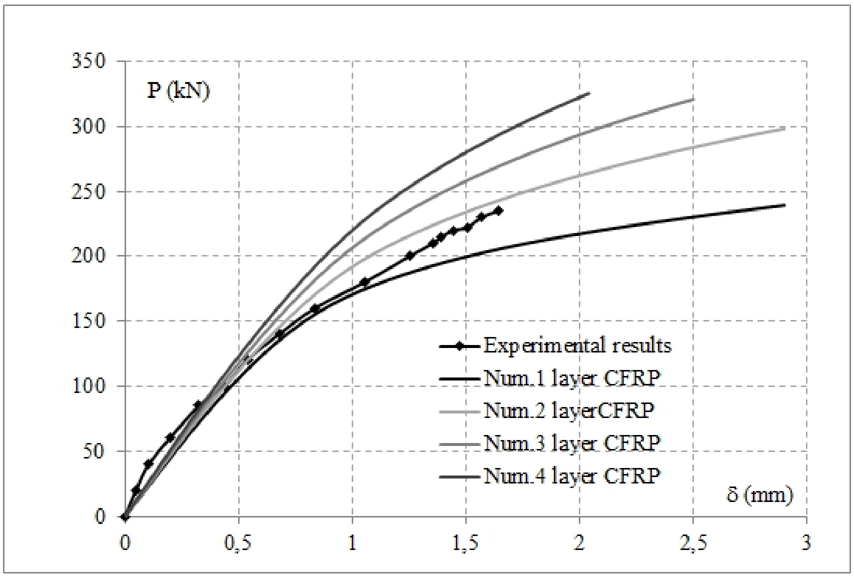


Figure 9: Load-displacement diagram of the compressed diagonal of CFRP reinforced panel: Variation of the number of CFRP layers.

the Young's modulus of the CFRP is increased, there is a slightly increase in the shear strength resistance. The greatest differences of maximum load and stiffness of retrofitted masonry are about 57% and 20%, respectively, and they correspond to four layers of the CFRP laminates externally bonded to the wall surfaces.

6 CONCLUSIONS

The shear behaviour of unreinforced masonry made of solid clay bricks present a high variability depending on the mechanical and geometrical properties of units and mortar, the characteristics of their interfaces, the geometrical arrangement of masonry, the boundary conditions

Table 5: Maximum load and stiffness of CFRP reinforced masonry for different values of k factor and CFRP layers

k factor	P_{max} (kN)	Stiffnes (kN/mm)
0.9	217	No influence
1.00	224	No influence
1.10	230	No influence
1.20	236	No influence
N^{o} of layers		
1	224.00	157.52
2	278.30	169.76
3	315.64	178.91
4	351.29	189.15

and the quality of workmanship. Generally, masonry capacity to support shear loading is bad presenting also a brittle failure mode. From the parametric study performed, it can be concluded that under shear loads, the Youngs modulus of the brick units is the most influential factor on load carrying capacity and stiffness of unreinforced masonry. An increase of 1.2 times of brick Young's modulus allows obtaining an increase of about 2.7% and 14% of the maximum load and stiffness of masonry, respectively. Retrofitting with CFRP increases ultimate load under in-plane shear, preventing the joints sliding. From the parametric analysis, it can be concluded that the increase in the number of CFRP layers is the most influential factor on load carrying capacity and stiffness of retrofitted masonry. Increases of about 57% and 20% of ultimate shear load and stiffness of reinforced masonry were obtained with four layers of CFRP.

Due to the cost, extension and complexity that experimental programs may have, it is important to have a numerical tool with which the unreinforced and FRP retrofitted masonry behaviour under different in plane stress states can be satisfactorily reproduced. Once the model has been adjusted, a numerical study allows the analysis of different loading conditions and repair or reinforcement assemblages, which is translated into smaller number of laboratory tests. At present, the authors are working on implementing a stochastic model in which the parameters herein analyzed are considered as random variables.

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