Study of the behaviour of reinforced masonry wallets subjected to diagonal compression through numerical modelling

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ABSTRACT: Shear walls are subjected to flexure and shear efforts in conjunction with compressive stresses associated to the gravity loads. In shear mode, diagonal cracks develop at the unit-mortar interface or both at the unit-mortar interface and through units as result of a biaxial tension-compression stress state, which in unreinforced masonry generally mean the collapse. The brittle failures of unreinforced masonry shear walls, which are more remarkable with high axial loads, may be prevented by the use of steel reinforcement. Diagonal compression tests allow obtaining a good prediction of the tensile strength of masonry walls in this biaxial tension-compression stress state. This paper aims to study the behaviour of reinforced masonry in diagonal compression tests through numerical modelling. A series of diagonal compression tests carried out on concrete block masonry with distinct types of reinforcement's arrangements are modelled using the software DIANA®. Results indicate that horizontal and vertical reinforcements applied in conjunct provide an increase on the shear strength and ductility. On the other hand, the application of horizontal reinforcements alone leads only to an increase of ductility.

Keywords: shear, diagonal compression tests, trussed reinforcement, numerical modelling.

1 INTRODUCTION

Over the last decades considerable research has been conducted on masonry structures. Masonry walls were mainly designed to bear gravity loads in spite of they have also an important role in improving seismic resistance and in global stability of masonry buildings since they can afford significant horizontal loads induced by earthquakes, which led to the idea that unreinforced masonry walls behave inadequately under seismic loading, being not allowed in zones of moderate to high seismic hazard. The brittleness of the failure of unreinforced masonry shear walls, which is more remarkable with high axial loads, may be reduced by the use of steel reinforcement. The role of the horizontal reinforcement on the shear resistance of masonry walls has been investigated in recent past in the perspective of the development of novel solution for reinforced masonry walls [1-4]. According to the findings of Haach et al. (2011) [3], it was seen that the influence of the horizontal reinforcement can be moderate if flexural mechanisms predominates over shear resisting mechanisms. Additionally, diagonal compression tests were carried out on concrete block masonry with distinct reinforcing configuration aiming at assessing their influence of the shear resistance when shear stress field is predominant [5]. In fact for this loading configuration shear stress predominates.

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Diagonal compression tests have been used by several authors to evaluate the shear behavior of masonry [6-9]. Recently Calderini et al. (2010) [10] have proposed the use of diagonal compression tests to derive the shear resisting properties in case of masonry in which crack patterns develop along the unit-mortar interfaces.

This paper has as the main objective to calibrate a numerical model based on the micro-modeling approach that describes the shear behaviour of the reinforced concrete block masonry. This is made by comparing the numerical results with the experimental results of diagonal compression tests on reinforced concrete masonry ones obtained by Vasconcelos et al. (2012) [5]. This model will be used further to perform sensitivity analysis to assess the influence of distinct parameters, like different vertical and horizontal truss type reinforcement percentages and distribution, on the shear resistance of concrete block masonry.

2 BRIEF DESCRIPTION OF EXPERIMENTAL TESTS

Diagonal compression tests were carried out on concrete block masonry of 60cmx60x10cm (reduced scale blocks) considering distinct reinforcement arrangements, see Figure 1. For the reinforced specimens nine reinforcement arrangements, corresponding mainly to distinct spacing for the vertical and horizontal reinforcements were adopted, see Table 1.

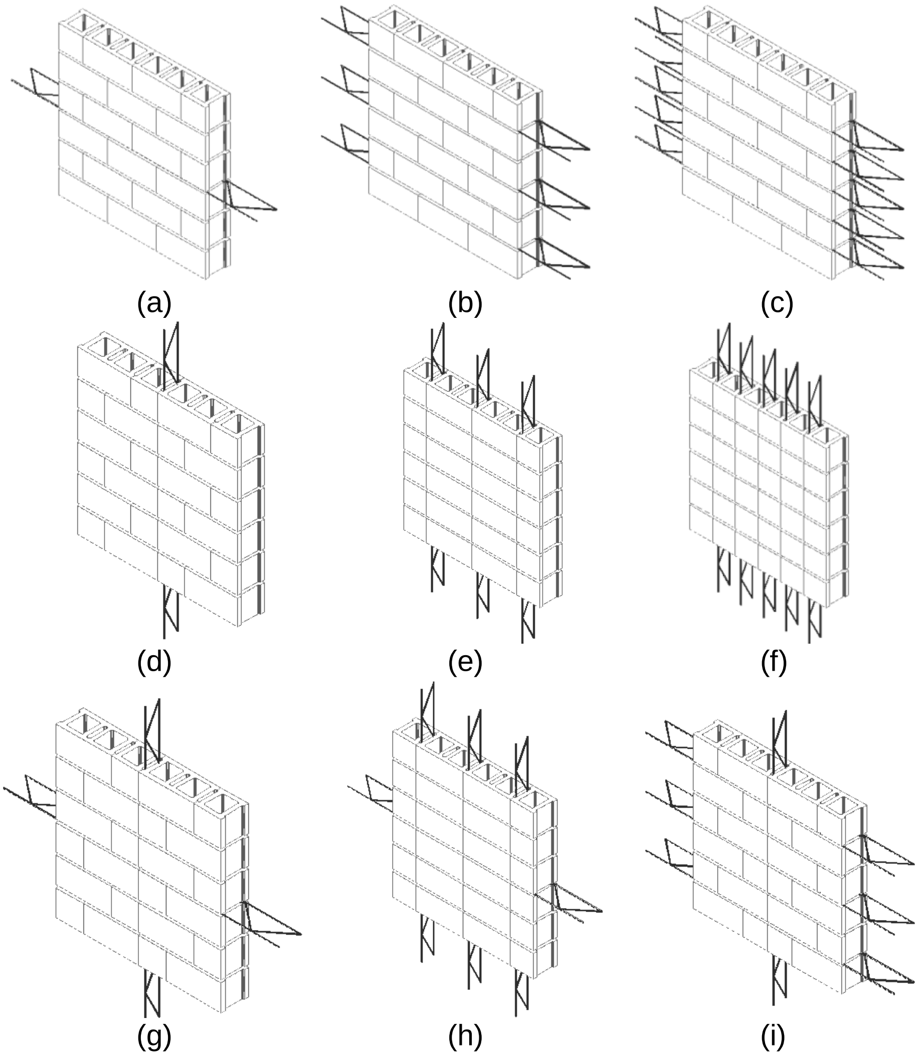


Figure 1. Reinforcement arrangements for the vertical and horizontal reinforcements; (a) specimen HRM1; (b) specimen HRM2; (c) specimen HRM3; (d) specimen VRM1; (e) specimen VRM2; (f) specimen VRM3; (g) specimen VHRM1 (h) specimen; VHRM2; (i) specimen VHRM3.

Table 1. Reinforcement percentage for each configuration.

Specimen tipology	Number of speciemens	ρ_h(%)	ρ_v(%)
HRM1	3	0.024	-
HRM2	3	0.071	-
HRM3	3	0.118	-
VRM1	3	-	0.042
VRM2	3	-	0.126
VRM3	3	-	0.209
VHRM1	3	0.024	0.042
VHRM2	3	0.024	0.126
VHRM3	3	0.071	0.042

The specimens have 600mm length, 605mm height, corresponding to six rows and 5 bed joints of about 8mm. The adopted dimensions for the specimens are related to the concrete blocks produced at half scale so that representative specimens could be found and with the recommended dimensions given by ASTM E519-02 (2002) [11]. The half blocks were also used in reduced scale masonry buildings tested at the shaking table of the National Laboratory of Civil Engineering (LNEC) [12] The truss type reinforcements have 60cm length and the spacing of the diagonal is 20cm. The latter dimension is approximately half of the commercial one so that the half scale of the blocks is considered.

The diameter of the longitudinal bars used for the horizontal direction was 3mm and for vertical direction was 4mm, according to the configuration adopted in the construction of the half scale masonry buildings [12]. The percentage of reinforcement corresponding to the adopted configurations is presented in Table 1. It should be noticed that some reinforcing configurations lead to considerable high reinforcement ratios.

As can be seen from Figure 1, the unreinforced masonry wallets were tested with traditional masonry bond with unfilled vertical joints to work as a reference. The specimens where only horizontal reinforcements were placed had also running masonry bond. On the other hand, the specimens with vertical reinforcements have vertical continuous joints so that the construction technology in real buildings can be simplified. In fact the blocks used are two cell blocks with end frogs, where vertical truss type reinforcements can be placed [1]. The vertical continuous joints, formed by the frogged ends of the concrete block units, have about 10mm and were filled with the general purpose mortar used for the laying of the units.

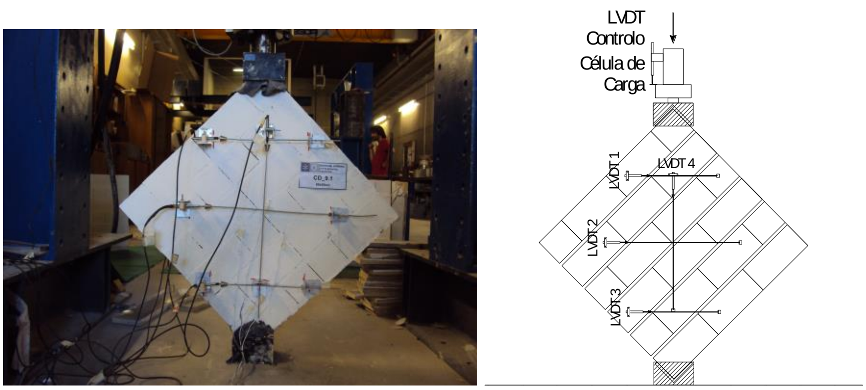


Figure 2. Instrumentation of the specimens.

The specimens were built with general purpose cement mortar in a proportion in volume of cement and sand of 1:3. The control of the construction quality was made by gathering mortar used in the construction of the specimens and further tested under uniaxial compression and flexure. The specimens were cured at laboratory environmental conditions in a place where the air relative humidity is almost constant and approximately equal to 65%.

The diagonal compression tests were carried out according to the recommendation of ASTM E519-02 (2002) [11]. The vertical load was applied by means of servo-controlled actuator, connected to a steel frame, with a load cell of 200kN, see Figure 2. The diagonal compression tests were carried out under displacement control, by means of a LVDT placed in the vertical actuator, at a rate of 2μm/s.

3 NUMERICAL MODELING

The numerical model applied to study masonry wallets subjected to diagonal compression was defined using the software DIANA® [13]. The micro-modeling approach was chosen for the simulation since it includes all the basic failure mechanisms that characterize masonry, enabling the detailed representation of resisting mechanisms of the masonry. The Newton-Raphson iteration procedure was used with displacement control, and an energy convergence criterion with a tolerance of 10^{-3} was adopted.

3.1. Geometry and Finite element mesh

For the numerical simulation a simplified micro-modelling approach was adopted. Thus, the finite element mesh was composed of continuum and interface elements to represent the masonry units and the masonry joints, respectively, see Figure 3. Aimed at foreseeing possible cracking passing through the units, potential vertical cracks were introduced at mid-length of the units. For the joints, six node interface elements with zero thickness and a 3-point Lobatto integration scheme were considered. Units were modelled with equivalent solid blocks to the actual hollow cell concrete blocks. In order to become the numerical model representative all properties of materials were defined by considering the gross area of the units. Units were modelled with two elements, which mean that each half of unit was modelled by one continuum element. Reinforcement was modelled through embedded bars, resulting in a slight increase in the stiffness of the finite element model. Reinforcement strains were computed from the displacement field of the continuum elements, which implies a perfect bond between the reinforcement and the surrounding material.

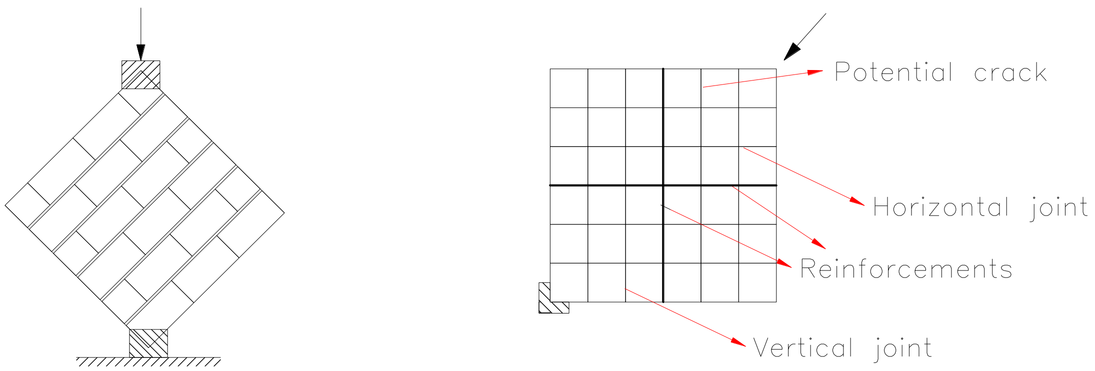


Figure 3. Example of mesh applied to the masonry wallets subjected to diagonal compression.

3.2. Loading and Boundary conditions

In the numerical modeling, monotonic loads were applied to the specimens. Specimens were not modelled in a diagonal position in order to make easy the pre-processing of the geometry. Therefore, the load was applied with displacement control at the corner of specimens in a direction of 45º. In the opposite corner of load application, nodes of the unit on the face of the specimen were encastred.

3.3. Material models and mechanical properties

In the micro-modeling approach all the constituent materials of the masonry walls, with distinct mechanical properties, are independently represented. Different material models were used to represent the behavior of concrete masonry units, vertical and horizontal joints and the potential cracks in the middle of the units. The most mechanical properties used in the description of the material models were defined by fitting the numerical to the experimental results obtained in the masonry wallets.

An interface cap model with modern plasticity concepts proposed by [14] and further enhanced by [15], was used for interface elements describing the masonry joints. The interface material model is appropriate to simulate fracture, frictional slip as well as crushing along the material interfaces, which are the possible failure modes of the masonry unit-mortar interfaces. Among the mechanical properties used for defining the yield functions in tension, compression and shear of the unit-mortar interfaces are the normal and transversal stiffness of bed joints (k_n = 180 N/mm^3 and k_s = 180 N/mm^3, respectively). The yield function with exponential softening for the tension cut-off model requires knowing the tensile bond strength of bed joints (f_t = 0.50 MPa) and the mode I fracture energy (G_{fI} = 0.03 N/mm). The shear behavior of the unit-mortar interfaces is represented by the Coulomb failure criterion. This function can be defined by knowing the cohesion (c = 0.60 MPa), friction coefficient (μ = 0.49), the dilatancy coefficient (tanψ = 0.0), and the shear fracture energy (G_{fII} = 5.00 N/mm). The behaviour of the masonry material in compression is modelled by a constitutive law composed by a parabolic hardening rule and a parabolic exponential softening branch ([14]). For the definition of this constitutive law the value of compressive strength (f_c = 7.50MPa) and the compressive fracture energy (G_c = 5.00 N/mm) are needed. Additionally, the parameter C_{ss} needed to take into account the contribution of shear stress to compressive failure (Css = 4.0), was defined by fitting the numerical to experimental results obtained in the masonry wallets.

In the case of the dry vertical joints, the shear behaviour was also modelled based on the Coulomb criterion, with null cohesion and a friction coefficient corresponding to the dry contact between two surfaces of concrete (μ = 0,65). Very low values of normal and transverse stiffness (4 N/mm^3) were considered, with zero tensile strength.

According to [14] it is useful to model potential cracks in units in order to avoid an overestimation of the collapse load and of the stiffness. Thus, potential cracks placed at the middle length of units were considered by using interface elements with a discrete cracking model. High stiffness should be considered for this interfaces, according to [14] (k_n = 10^6 N/mm^3 and k_s = 10^6 N/mm^3, respectively). In addition, an exponential softening behavior was adopted for the tensile behavior of these interfaces with the tensile bond strength (f_t = 6.25 MPa) and the mode I fracture energy (G_{fI} = 0.06 N/mm). The constitutive law for discrete cracking in DIANA® is based on a total deformation theory, which expresses the tractions as a function of the total relative displacements.

The non-linear behavior of the concrete masonry units was represented by a Total Strain Crack Model based on a fixed stress-strain law concept available in the commercial software program DIANA®. It describes the tensile and compressive behavior of the material with one stress-strain relationship in a coordinate system that is fixed upon crack initiation. Exponential and parabolic constitutive laws were used to describe the tensile and compressive behavior of concrete masonry units, respectively. The mechanical properties needed to describe this material model are the elastic modulus of concrete units (E = 15.00 GPa), the Poisson's ratio of concrete units (v = 0.20), the tensile and compressive strength of concrete units (f_{tu} = 6.25 MPa and f_{cu} = 12.13 MPa), the fracture energy of units under tension (G_{fuI} = 0.06 N/mm) and the shear retention factor (β = 0.01).

An elasto-plastic model based on the yield criterion of Von Mises was adopted to describe the behaviour of the reinforcement considering the yield stress equal to 580 MPa and the Young's modulus equal to 196 GPa. These properties were obtained from tensile tests carried out on reinforcements [16]. As the reinforcement elements overlap the interface elements representing the masonry joints, and thus have traction components in the same directions as the interface elements (normal and shear components), a 'free length' (thickness of the joints) is needed in order to properly account for the stiffness of the interface crossed by the reinforcement. Reinforcement considerably increases the stiffness of the interface elements and the additional normal and shear stiffness of the interface elements crossed by the steel reinforcements is given respectively by Eq. 1 and Eq. 2:

$$k_n = \frac{E_s}{l_{fr}} \quad (1)$$

$$k_s = k_t = \frac{E_s}{2l_{fr}} \quad (2)$$

where, E_s is the elastic modulus of reinforcements and l_{fr} is the thickness of mortar joints.

It should be stressed that the presence of reinforcement leads to a significant increase of the elastic stiffness of the interfaces. As the stiffness attributed to the interfaces is much larger than the stiffness attributed to the masonry joint, the global non-linear problem becomes ill-conditioned. The number of iterations needed to achieve convergence, and consequently the computational effort, increase.

4 RESULTS

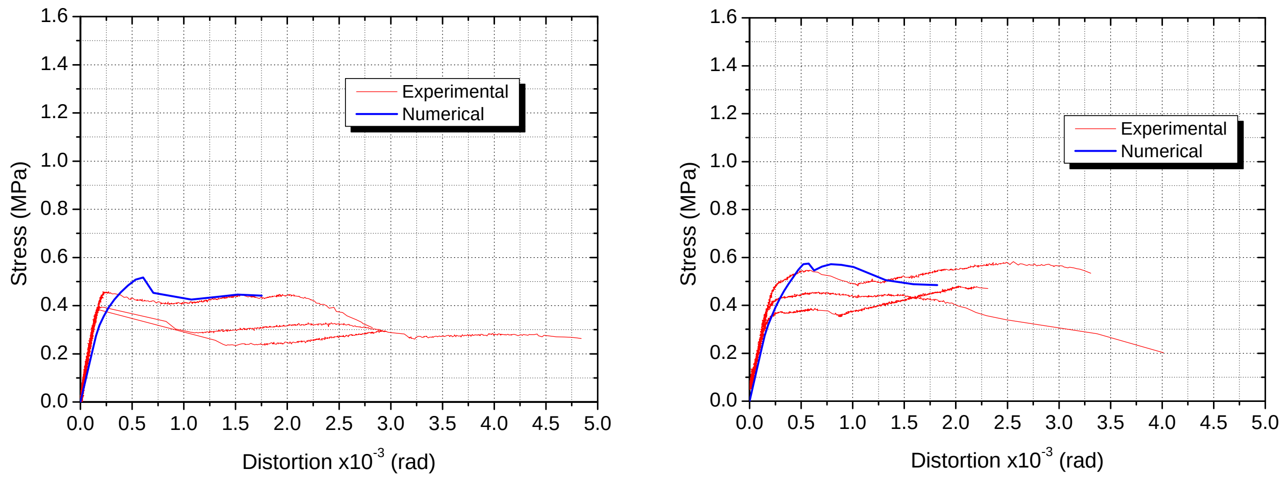
Table 2 presents a comparison between numerical and experimental results. In general numerical modelling represented very well the behaviour of masonry specimens tested in laboratory. In terms of shear strength, the differences between numerical and experimental results were not higher than 30 % considering average values.

Table 2. Comparison between Experimental and Numerical results.

Specimen	Exp. shear strength (MPa)	Num. shear strength (MPa)	Num./ Exp.	Exp. shear stiffness (GPa)	Num. shear stiffness (GPa)	Num./ Exp.
Unreinforced	0.41 (9.9 %)	0.52	1.27	2.59 (8.0 %)	1.82	0.70
HRM1	0.51 (13.2 %)	0.57	1.12	2.41 (2.8 %)	1.87	0.78
HRM2	0.73 (13.9 %)	0.64	0.88	2.65 (6.8 %)	1.73	0.65
HRM3	0.87 (4.1 %)	0.85	0.98	2.77 (11.8 %)	1.99	0.72
VRM1	0.91 (17.8 %)	0.64	0.70	2.89 (7.0 %)	1.83	0.63
VRM2	1.19 (5.0 %)	1.12	0.94	3.39 (7.1 %)	2.98	0.88
VRM3	1.06 (38.7 %)	1.35	1.27	3.57 (5.5 %)	2.80	0.78
VHRM1	0.91 (0.7 %)	0.80	0.88	2.51 (2.2 %)	1.93	0.77
VHRM2	1.19 (8.5 %)	1.13	0.95	3.21 (3.8 %)	2.99	0.93
VHRM3	0.86 (10.4 %)	0.79	0.92	2.56 (8.6 %)	1.93	0.75

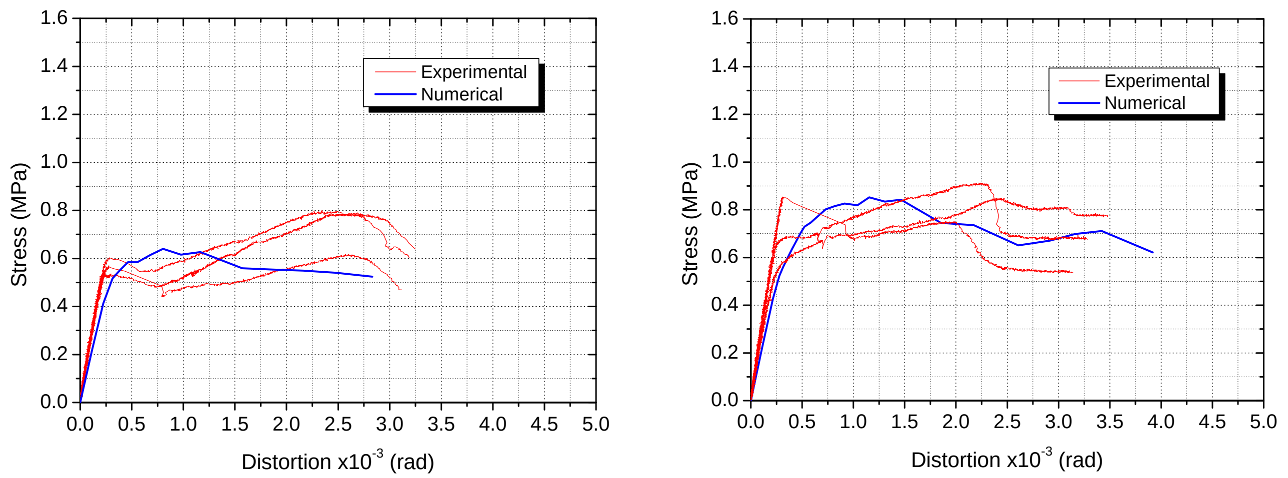
If the coefficient of variation of the experimental results was considered these differences were not higher than 17 %. A similar behaviour could be observed for shear stiffness calculated from the Shear stress *vs.* Distortion diagrams. Numerical shear stiffness was lower than experimental ones for all masonry specimens. This behaviour occurred probably because normal stiffness of mortar joints is different under tension and compression. Under tension the normal stiffness of mortar joints is very lower than under compression [17]. However in the numerical modelling was considered a constant normal stiffness for all mortar joints. Figure 4 to Figure 8 presents a comparison between numerical and experimental Shear stress *vs.* Distortion diagrams. These diagrams confirm the previous comments based on results of Table 2. Numerical modelling represents the experiments in a good way in terms of shear strength, shear stiffness and ductility.

Crack patterns with one main vertical crack in the center of specimen due to the shear stresses following a stair-case pattern were observed in all numerical specimens as in case of the experiments in the masonry wallets, see Figure 9. All numerical specimens presented small stresses in the reinforcements up to the maximum shear stress, see Figure 10, after which presents a significant increase. This means that the reinforcements only contribute for the resistance very near to the peak force after the cracking is initiated. Results indicated also that reinforcements had a significant influence in post-peak behaviour increasing the ductility of masonry wallets. This is particularly evident in case of experimental specimens with horizontal reinforcement. Numerically, the introduction of the vertical reinforcements do not results is so fragile behaviour (Figure 7a).



(a) (b)

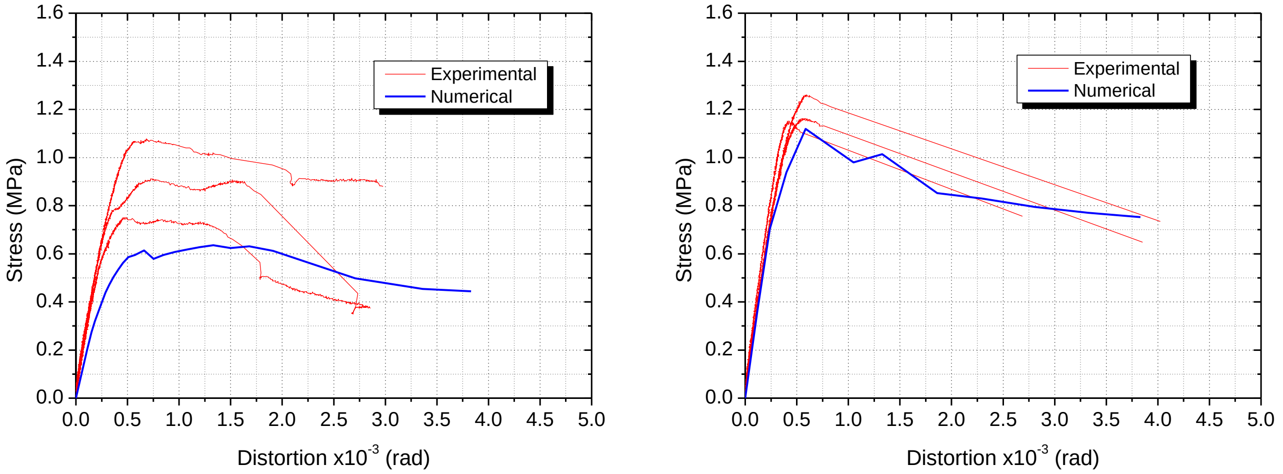
Figure 4. Stress *vs.* Distortion diagrams: (a) Unreinforced masonry and (b) HRM1.



(a) (b)

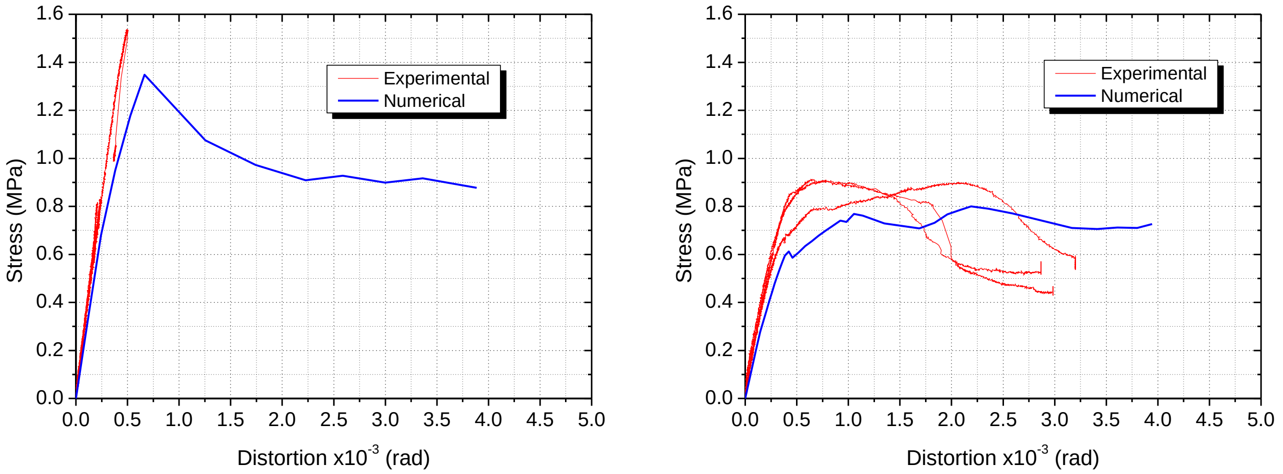
***Figure 5.** Stress vs. Distortion diagrams: (a) HRM2 and (b) HRM3.*

Figure 11 presents the comparison of Stress *vs.* Distortion diagrams for specimens VRM1 and VRM2 with and without reinforcements. It is observed that the increase on the lateral strength depends on the percentage and spacing of the vertical reinforcement.



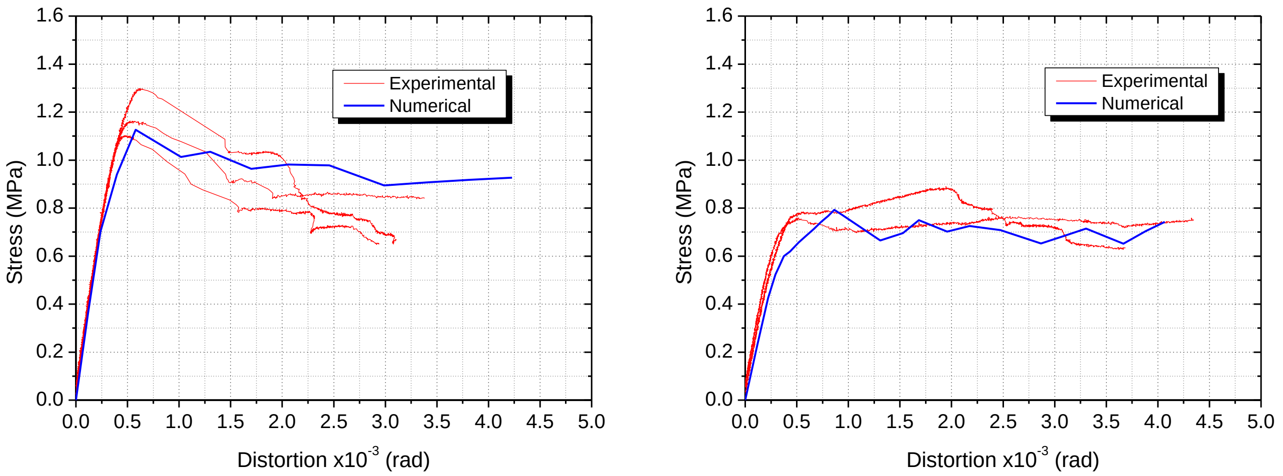
(a) (b)

Figure 6. Stress *vs.* Distortion diagrams: (a) VRM1 and (b) VRM2.



(a) (b)

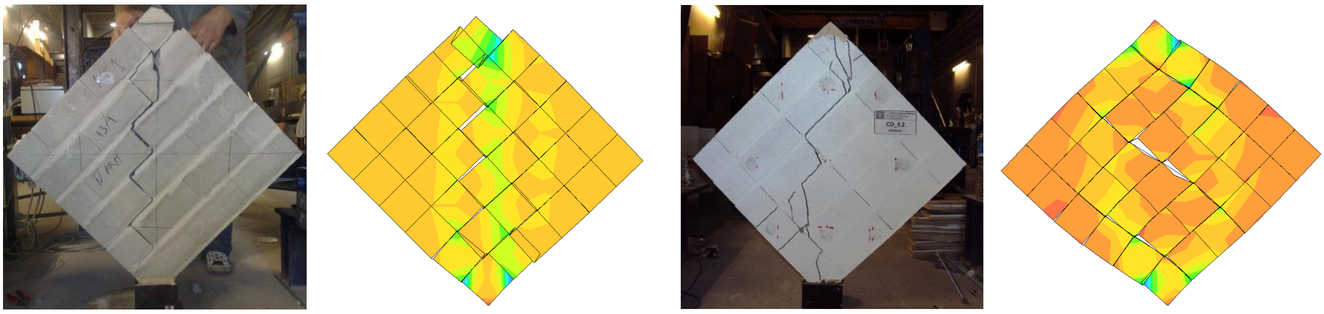
Figure 7. Stress *vs.* Distortion diagrams: (a) VRM3 and (b) VHRM1.



(a) (b)

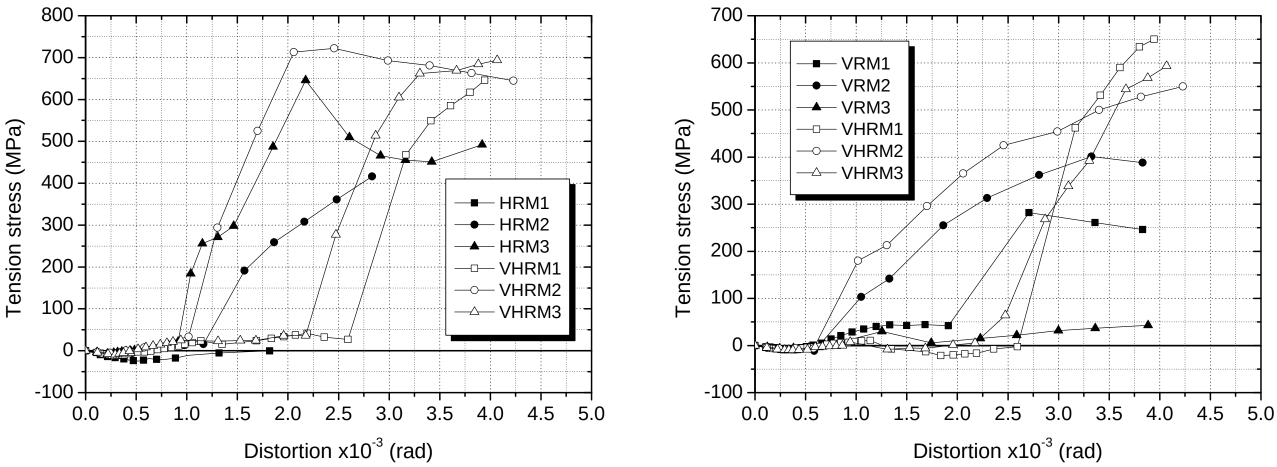
Figure 8. Stress *vs.* Distortion diagrams: (a) VHRM2 and (b) VHRM3.

The increasing of shear strength of specimens HRM1, HRM2 and HRM3 and the specimens VRM1, VRM2 and VRM3 should be also associated to the increase of the strength of some mortar joints due to the presence of reinforcements (see equations 1 and 2) leading to improvement of the shear strength of those masonry wallets. Besides, the masonry bond pattern had a significant influence in increasing of the shear strength of models as can be observed comparing unireinforced masonry wallet and specimens VRM1 and VRM2 without reinforcement.



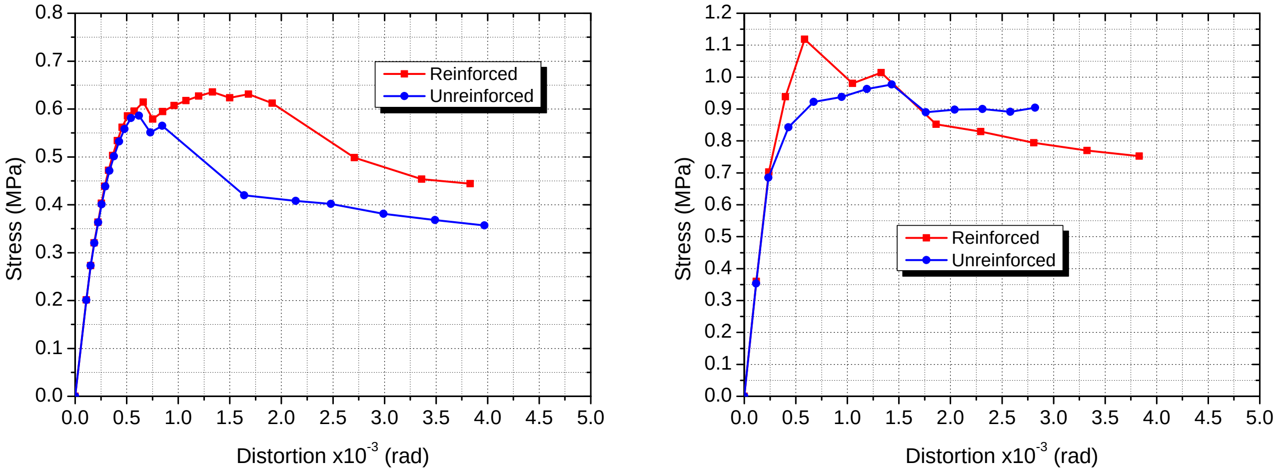
(a) (b)

Figure 9. Failure modes: (a) Unreinforced masonry and (b) VRM2.



(a) (b)

Figure 10. Tension stress in reinforcement inthe center of specimen: (a) Horizontal and (b) Vertical.



(a) (b)

Figure 11. Comparison of Stress *vs.* Distortion diagrams for specimens with and without reinforcements: (a) VRM1 and (b) VRM2.

5 CONCLUSIONS

The study presented a numerical modelling of diagonal compression test of reinforced concrete block masonry wallets. Due to the absence of characterization of unit-mortar interfaces, the most material parameters were adopted considering the fitting of numerical and experimental results of the diagonal compression tests. Numerical modelling represented very well the experiments in terms of failure mode, shear strength, shear stiffness and ductility. Numerical results indicated that reinforcement presents low strain before the peak, indicating that it is active only after the crack initiation that occurs close to the peak load. It should be stressed that reinforcements and in particular the horizontal reinforcements contribute for the increase on the ductility. Besides, masonry bond pattern has a significant influence in behaviour of masonry wallets.

Finally, based on the comparison between numerical and experimental main results, it can be concluded that the numerical model is able to reproduce the experimental mechanical behavior of reinforced and unreinforced masonry wallets under diagonal compression load, meaning that it is suitable to be used on a future parametric analysis.

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