

Determination and validation of input parameters for detailed micromodelling of partially grouted reinforced masonry walls

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ABSTRACT: Partially grouted reinforced masonry walls constructed with multi-perforated clay bricks are widely used for low-rise constructions in some seismic regions and, consequently, it is necessary to adequately comprehend its behavior, and to develop suitable tools for reproducing its response and estimating its shear strength. Aimed at obtaining a better understanding of the shear behavior of this masonry typology, this paper presents a comprehensive experimental program carried out in order to characterize the mechanical properties of the constituent materials and the response of the unit-mortar interface. Special attention is given in determining the shear response of small assemblages. Then, experimental results are used to create numerical simulation models by using the detailed micro-modelling approach. Finally, the obtained experimental data and simulation knowledge are applied in the reproduction of the shear response of two pairs of full-scale walls, tested in the laboratory under cyclic lateral loading. The comparison between numerical results and experimental response provides significant conclusions.

1 INTRODUCTION

Reinforced Masonry (RM) walls are commonly used in low-rise construction mainly in seismic regions of South and Central America, South-East Asia, Middle East, and South-Eastern Europe. As it is well known, RM can be either fully grouted (FG) or partially grouted (PG). Fully grouted means that all hollows of masonry units are completely grouted, whilst partially grouted implies that only hollows with presence of vertical steel bars are completely grouted.

In-plane seismic response of PG-RM walls depends on the geometry of the walls, mechanical characteristics of its constituent materials and bonding patterns between them, as well as on the boundary conditions. In the case of masonry constructed with multi-perforated clay bricks, its behavior also depends on the amount of fresh mortar that penetrates into the vertical perforations of bricks, originating stiff internal cores once it has hardened, and producing an interlocking mechanism (Zepeta *et al.* 2000; Gabor 2006; Abdou *et al.* 2006). In fact, the presence of this mechanical interlock increases the joint stiffness (Abdou *et al.* 2006), but would be dependent of the thickness and resistance of interior and exterior brick walls. The above-described mechanism could present a significant influence in the shear response of PG-RM elements or structures but, however, its influence has been scarcely investigated both experimentally and numerically.

Several experimental works have been carried out on the seismic behavior of partially grouted masonry walls constructed with multi-perforated clay bricks. A summary with relevant information of them can be found in Zepeda *et al.* (2000). However, only limited experimental evidence has been reported on shear response along mortar bed joints considering multi-perforated brick masonry. Gabor *et al.* (2006) evaluated the shear behavior parameters of small hollow brick masonry specimens, finding that interface behavior is governed by the blocking mechanism between the internal walls of bricks and the mortar cores. The obtained parameters were used for numerically reproducing the case of diagonal compression test of masonry panels. A similar investigation was carried out by Fouchal *et al.* (2009). In this campaign, the results showed that the presence of mortar cores can affect the softening behavior due to

the blocking mechanisms that takes place at the brick-mortar interface.

Due to the diverse local mechanisms involved in shear behavior of PG-RM walls, a numerical strategy capable to adequately model all the different involved items (units, mortar and interfaces) should be used. In this context, detailed micro-modeling is the most appropriate numerical approach. However, this approach requires a substantial computational effort along with a large amount of input parameters needed for the proper reproduction of the phenomena governing the structural behavior of PG-RM walls. Some attempts for simulating the shear behavior of multi-perforated clay brick masonry have been undertaken using detailed micro-models (Gabor *et al.* 2006; Fouchal *et al.* 2009; Rahman & Rueda 2014). Nevertheless, all these works consider that nonlinear behavior of masonry is concentrated in the interfaces and in the mortar, do not considering the brick tensile failure observed in several post-earthquake damage observations (see e.g. Astroza *et al.* 2012; Valdebenito *et al.* 2015) and in experimental tests (see e.g. Tomazevic 2009).

Firstly, this paper presents an experimental campaign aimed to characterize the properties of the constituent materials (bricks and mortar), of the unit-mortar interface, and of the composite material. To this end, compression, tension and shear tests were carried out on small masonry assemblages constructed with multi-perforated clay bricks. Later, the experimental data obtained are directly used as input parameters in the reproduction of triplet tests and of diagonal compression tests by means of detailed micro-modelling. Secondly, the paper presents the results of two full-scale PG-RM walls which were tested in the laboratory under cyclic lateral loading. Both tests are numerically reproduced using the simulation techniques validated in the first part. The suitability of the applied data and the accuracy achieved by the proposed detailed micro-modelling in reproducing the structural response of PG-RM walls is observed as satisfactory.

2 EXPERIMENTAL CHARACTERIZATION AND MODELLING

2.1 Brick

The multi-perforated clay bricks used in this research present a nominal dimensions of 290x140x112 mm³. Their compressive properties were determined by means of six tests performed according to NCh167.Of2001 (INN 2001), obtaining an average compressive strength of f_b =33.7 N/mm², and an average Young's modulus of E_b =7714.8 N/mm². The percentage of voids and water absorption of bricks were also obtained according to NCh167.Of2001 (INN 2001), resulting in an average of 50.5% and 10.4% respectively.

As commented in previous section, some damages observed in recent earthquakes have shown that diagonal cracks (x-shaped) are develop across both units and brick-mortar interfaces. In consequence, it becomes also necessary to characterize the tensile strength of the units, which in this case was indirectly obtained by means of three-point load tests (Fig. 1). The value of the tensile strength f_{tb} has been estimated from the flexural one f_{xb} by using the equation provided in the Model Code 90 (CEB 1990) (Eq. 1), where *h* is the height of the specimen tested. The three-point load test were applied to three equal bricks, obtaining an average maximum applied force of 7869.2 N with a CV=14.7%, which represents a flexural strength of $f_{xb} = 1.248$ N/mm². The appliance of Eq. (1) provides an average tensile strength of $f_{tb} = 0.77$ N/mm².

The validation of the numerical strategy selected to reproduce the brick fragile response in tensile is carried out through the simulation of the performed test. One brick finite elements model is created by means of 30 8-node plane stress elements (codified as CQ16M in Diana 9 – TNO Diana 2014), defining a reasonable mesh size in order to be used in complete walls simulation. A total strain rotating crack model is selected, using the experimentally obtained parameters of modulus of elasticity, and compressive and tensile strengths. The complete reproduction of the tensile failure requires to consider the energy dissipation occurred. Different values of tensile fracture energy, ranging from 0.015 to 0.03 N.mm/mm², are applied through an exponential softening curve, obtaining that the value of 0.02 N.mm/mm² is the one that provides a better fitting to the average experimental maximum load (Table 1).



Fig. 1 – Three-point load test: (a) scheme and (b) typical failure mode.

Table 1 - Numerically obtained brick failure load for different fracture energy values.

	Experimental	Brick tensile fracture energy, $G_{fl,b}$ (N.mm/mm ²)						
_	average	0.015	0.0175	0.02	0.0225	0.025	0.0275	0.03
3-point test brick	7869.2	7497.32	7691.42	7826.44	7987.25	8218.63	8391.48	8575.35
failure Load (N)		(-4.73%)	(-2.26%)	(-0.54%)	(1.50%)	(4.44%)	(6.64%)	(8.97%)

2.2 Mortar

The mortar used was obtained from pre-mixed commercial cement typically used in the Chilean masonry constructions. Its compressive properties were obtained from compressive tests carried out on cylinders with 50 mm of diameter and 100 mm of height, according to NCh158.Of67 (INN 1999). From six tests, the average elastic modulus obtained after 28 days was $E_m = 4460 \text{ N/mm}^2$ (CV=10.8%) and the average compressive cylinder strength was $f_m = 12.6 \text{ N/mm}^2$ (CV=3.0%).

Twelve flexural tests were carried out on prismatic specimens $40x40x160 \text{ mm}^3$, where an average flexural strength of $f_{xm} = 5.1 \text{ N/mm}^2$ (CV=9.1%) was obtained. As previously performed for the multiperforated clay bricks, tensile strength of mortar f_{tm} has been estimated from the flexural one f_{xm} by using Eq. (1), resulting in an average value of $f_{tm} = 2.25 \text{ N/mm}^2$.

2.3 Brick – Mortar interface: Tensile

Direct tensile tests were carried out in order to characterize the tensile bond in normal direction, which were performed on specimens composed by two bricks connected by a single mortar bed joint. A deformation rate of 0.5 μ m/s was applied to all tests, using four displacement transducers (LVDT) placed at half-height of each brick in order to accurately measure the displacement (Fig. 2a).



Fig. 2 - *Direct tensile test: a) test setup and instrumentation, (b) typical failure mode observed during the tests, and (c) obtained bond tensile strength curves.*

The bond tensile strength of the interface f_t was found to vary from 0.1 to 0.16 N/mm² with an average of 0.13 N/mm² (CV=20.3%). The typical failure mode observed during the tests is shown in Fig. 2b. The analysis of the tensile stress-elongation curves shows a considerable scatter on the bond tensile strength, but very similar softening branches. The descending curves show some irregularities due to the non-uniform crack opening (Fig 2c), as already reported by other researchers (Almeida *et al.* 2002).

Correspondent tensile fracture energy G_f^I values determined according to the net contact area range from 0.0009 to 0.002 Nmm/mm².

2.4 Brick – Mortar interface: Shear

The shear behavior presented by the multi-perforated brick-mortar interface is here determined by means of shear triplet tests according to NCh167.Of2001 (INN 2001). For that, 16 triplet specimens consisting in three units and two mortar joints were constructed and tested according to the configuration presented in Fig. 3a. All specimens were built with 10 mm width mortar joints. Five different pre-compression levels were applied on specimens in order to produce the normal stresses of 0, 0.6, 0.9, 1.2 and 1.5 N/mm² (determined on the net brick area), covering a reasonable compression work range for masonry walls in real constructions. Compression stress levels were maintained constant during the tests by means of a self-equilibrated steel frame (Fig. 3a). Rubber pieces were used at the extremities of the samples to avoid the concentration of stresses caused by hard contacts. A set of LVDT's was used in both sides of specimens to appropriately register the behavior of the unit-mortar interface under shear stresses. It should be noted that the specimens tested under null compression were performed without the confinement steel frame.



Fig. 3 – Triplet shear test setup (a), and failure mode for normal stress of 1.2 MPa (b).

Different kinds of failure mode were obtained depending on the normal confinement level. In the specimens under null compression stress was observed an interface failure, do not affecting significantly the bricks. The increase of the compression stress level caused a combined failure of fracture planes along mortar-brick interfaces, cracks through the mortar layer and, in some cases, cracking and crushing of interior walls of bricks. For elevated levels of normal compression, failure mode is mainly based on the crushing of the interior walls of the bricks, producing in some cases a brittle and sudden failure mode that even caused the spalling of the external walls of the bricks (Fig. 3b).

As can be observed in Fig 4a, which presents the shear stress-displacement diagrams obtained from the triplet tests, all curves show a quite similar trend regardless of the normal compression stress. Regarding to the initial stiffness, all specimens showed a rigid response up to the maximum shear stress, except for the specimens under null compression due to the above mentioned absence of the confinement steel frame. Following the maximum shear stress, a gradual degradation of the interface shear response can be clearly observed. Only in the cases of normal stress of 1.5 MPa, registered curves confirm the sudden and brittle failure mode already observed during the tests. Therefore, and contrarily to the fragile behavior presented in triplet tests when solid bricks are used (see e.g. van der Pluijm 1993), the response in multi-perforated clay brick masonry is characterized by a long softening behavior, progressively reducing the maximum shear stress resisted by the interface as the slip increases. As commented by Fouchal et al. (2009), this behavioral difference could be explained by the presence of mortar cores at the unit-mortar interface, producing a combination of the frictional response with the step by step failure of the internal walls of the bricks and the mortar. The linear correlation between the maximum shear stress and the normal compression applied in terms of Mohr-Coulomb frictional criterion is shown in Fig 4b. From this relationship, a mean value for cohesion of c = 1.052 N/mm² and an internal friction angle of $\phi = 37.5^{\circ}$ (tan $\phi = 0.767$) are derived.

The interfaces between bricks and mortar are usually faced through Mohr-Coulomb frictional models (Gabor *et al.* 2006; Haach *et al.* 2011; Rahman & Ueda 2014; among others), which become basically defined by a cohesion value and a friction angle. For a constant normal pressure, and out of considering dilatancy effects, the model provides a constant maximum shear stress, and consequently, do not satisfactorily fitting the experimentally obtained response. In order to successfully reproduce the long

softening branch obtained in the shear stress *vs* shear slip diagram, the value of the frictional cohesion is modified depending on the shear sliding as it is shown in Fig. 5a. The points of these relation are obtained from the experimental results by subtracting the pure friction component of the maximum shear (average of 0 and 0.6 MPa normal compression cases), and implemented into the model through the cohesion hardening option available in the software Diana (TNO Diana 2014).



Fig. 4 – Shear test results: (a) stress-displacement curves and (b) relation between maximum shear stress τ and normal applied stress σ_n .



Fig. 5 – Triplet test simulation: (a) frictional cohesion vs shear sliding curve and (b) FE model.



Fig. 6 – Comparison between numerical and experimental shear curves obtained in the triplet tests.

The validation of the proposed model is performed trough the reproduction of the triplet test by means of a detailed micro-model, which individually considers the response of the mortar, the bricks and the interfaces (Fig. 5b). In order to clearly apprise the interface response, bricks and mortar are considered as linear. The numerically obtained results for the normal compressions of 0, 0.6, and 1.2 are compared with the experimental ones in Fig. 6, showing a highly satisfactory degree of accuracy. Consequently, it is assumed that the proposed model is able to successfully reproduce the complex shear response presented in the bed joints of multi-perforated brick masonry.

2.5 Diagonal compression test: Experimental and numerical response

The shear response of the masonry composite was obtained by means of five diagonal compression tests, carried out according to NCh2123.Of97 (INN 2003). The diagonal load was introduced by means of a 60 t hydraulic jack connected to a self-equilibrated steel frame, whilst the displacements measurements were performed by two displacement transducers (LVDT) located at each side of the specimen. Fig. 7a shows the test configuration. Fig. 7b shows the load-displacement diagrams obtained for the tested specimens. In general, the failure mode consisted in a large crack oriented in the load direction, which develops across both mortar joints and bricks, as can be observed in Fig. 8a.



Fig. 7 – Diagonal compressive test configuration (a), and obtained results (b).

Previously validated strategies for reproducing both brick and brick-mortar horizontal interface are applied to simulate the diagonal compression test. The load is introduced as an imposed displacement in one corner whilst the opposite one is fixed. A part from the data obtained in the experimental campaign, the reproduction of the masonry composite requires to appropriately define some additional parameters (table 2). The gap formation value of 0.13 N/mm² corresponds to the value experimentally obtained in the direct tensile stress resisted by the brick-mortar interface. The frictional properties of the flat vertical joint were assumed according to the experimental results obtained by Alcaino (2007) on similar bricks and mortar. For horizontal joints, a post opening shear stiffness is assumed in order to consider the key effect of the mortar cores when separation occurs. Its value of 959.87 N/mm³ is mechanically determined by assuming an average opening (and consequently defining the shear deformation height of the cores) of 1 mm. For vertical joints a null value of post-opening shear stiffness is assumed according to its plane face. Poisson coefficient and tensile fracture energy of mortar are selected according to the recommendations performed by Drougkas *et al.* (2015). Table 2 summarizes the material parameters used in the numerical model.



Fig. 8 – Experimentally (a) and numerically (b) obtained crack pattern in diagonal test.

The numerically obtained load-displacement curve for the diagonal compression test is presented in Fig. 7b, showing and excellent agreement to the experimental results in terms of average stiffness and in the first part of the damaged response. Around 95 kN load, the masonry stiffness decrease due to the start of elements damage and significant joints slipping, being accurately reproduced by the proposed numerical model. On the other hand, the maximum load values cannot be reproduced with same precision since the numerical model leads to unstable calculation when damages in bricks and mortar are highly significant. Numerical crack pattern also presents an excellent agreement to experimental

ones (Fig. 8b), showing the same diagonal crack that involves both bricks and mortar cracking, and the slipping of the joints.

Elements	Bricks	Mortar	Grout	
Modulus of elasticity, $E(N/mm^2)$	7714.8	4460	21763	
Poison coefficient, v	0.15	0.15	0.2	
Tensile strength, σ_t (N/mm ²)	0.77	2.25	3.62	
Tensile fracture energy, G_1 (N.mm/mm ²)	0.02	0.0716	0.1	
Compression strength, σ_c (N/mm ²)	33.7	12.6	24.15	
Compression fracture energy, G_c (N.mm/mm ²)	14.35	14.91	14.91	
Interfaces	Horizontal Brick-mortar	Vert. Brick-mortar Brick- grout	Grout-mortar Concrete- mortar	
Cohesion, C (N/mm ²)	1.052	0.6	0.819	
Friction angle, ϕ (°)	0.767	0.767	0.6	
Tensile strength, σ_t (N/mm ²)	0.13	0.13	1.125	
Post opening shear Stiffness, $K_{s,o}$, (N/mm ³)	959.87	0	0	

Table 2 – Values of mechanical properties used in the detailed micro-model of the walls.

3 SHEAR WALLS. EXPERIMENTAL AND NUMERICAL

Four full-scale walls using hollow brick units were tested under in-plane cyclic lateral loading representing two specimens for two different amounts of horizontal reinforcement. The wall dimensions were 1990x2012 mm² given a height-to-length (aspect) ratio of 0.97. Walls were designed and constructed with base and top oversized heavily reinforced concrete beams to connect them to the loading frame. Specimens were constructed by an experienced construction worker, following Chilean traditional techniques, and employing running bond pattern with mortar joints of 10 mm thick. The vertical boundary reinforcement was 2 Ø22 mm bars for both specimens, whilst the horizontal reinforcement consisted of shaped prefabricated ladder-type reinforcement with 4.2 mm diameter, typically used in PG-RM structures in Chile. In case of walls 01/02, the horizontal reinforcement ratio was of 0.05% (5 rows) whereas for walls 03/04 was of 0.07% (7 rows). In Fig. 9a can be distinguished the distribution of vertical and horizontal reinforcement of walls 01/02.

The walls were subjected to in-plane controlled displacement cyclic loading with a cantilever-type boundary condition. An axial stress of 0.55 MPa was applied through a vertical actuator and was kept constant during the tests in order to reproduce the compressive stress of a ground floor wall in a three-story building with two concrete slabs and a light roof. The typical failure mode observed during the tests can also be distinguished in Fig. 9a. The envelop curves obtained from the cyclic load-displacement registers (for both first and inverse directions) are presented in figure 10. As can be clearly apprised, the increase of the horizontal reinforcement amount produces an increment of both ultimate load and maximum lateral displacement.

The shear tests of the walls are reproduced by means of the detailed micro-modeling strategy that has been developed, tested and validated in previous research stages. A full scale model of the masonry wall and the support concrete beams is performed through 8-node plane stress elements, placing frictional interface elements between all constitutive materials (Fig. 9b). Reinforcement bars are modeled as bar elements with coincident nodes to the plane stress ones, placed in the middle of the horizontal mortar joint or the grout column. Steel reinforcement bars, using the next properties for horizontal (h) and vertical reinforcements (v): -modulus of elasticity: 174.7 Gpa (h) and 224.0 GPa (v), yielding stress of 490.5 MPa (h) and 465.0 (v), maximum stress of 642.0 Mpa (h) and 750.7 MPa (v), and a ultimate strain of 0.01. Grout compression strength and modulus of elasticity were measured in the laboratory, whilst the rest of the properties were derived from them by usual relations find in the bibliography. Table 2 summarizes the values of the mechanical properties used in the numerical modelling of the walls.

First, the walls are loaded in vertical direction with the applied load and the dead weight, to latter introduce the horizontal loads as imposed displacement steps of 0.025 mm. Numerical obtained force-

displacement diagrams for both reinforcement configurations are presented in Fig. 10. As can be observed, the proposed strategies for detailed micro-modelling present an excellent accuracy to the experimental results, moreover when analyzing the complexity of the mechanisms being reproduced. The loading curves and maximum loads are accurately reproduced. Maximum displacement of walls 01/02 is slightly underestimated but, on the other hand, the maximum displacement prediction in walls 03/04 is excellent. Additionally, the obtained crack patterns also present a very good agreement (Fig. 9b), showing the same position of the main crack, and the same failure mechanism based on the combination of mortar and brick cracking, and joints slipping.

Fig. 9 – Experimentally (a) and numerically (b) obtained crack pattern in wall 01 test.

Fig. 10 – Experimental and numerical shear response of the walls.

4 CONCLUSIONS

The mechanical response of the constituent elements and interfaces involved in multi-perforated bricks masonry has been experimentally characterized through a complete set of experiments. The mortar cores created inside the bricks present a significant influence in the shear response of the horizontal joints, also promoting the cracking of the bricks. Numerical strategies have been proposed and validated in order to adequately reproduce both phenomena in the base of the detailed micro-modelling approach. The application of these techniques has allowed to successfully reproduce the shear behavior exhibit by the masonry composite in the diagonal compression test by directly applying the experimental results, without necessity of calibration parameters. Furthermore, their applicance in order to reproduce the complex shear response of a partially grouted and reinforced masonry walls also provided highly satisfactory results. Consequently, it is concluded that the proposed numerical approach based in the detailed micro-modelling can successfully reproduce the shear response of multi-perforated clay brick masonry, thus emerging as a reliable tool in order to perform further studies on the structural response presented by this masonry typology.

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