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COMPRESSIVE AND SHEAR BEHAVIOUR OF MASONRY PANELS: EXPERIMENTATION AND NUMERICAL ANALYSIS

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Abstract. *The compressive and shear behavior of masonry is here studied both experimentally and numerically. An experimental campaign has been carried out on 9 square-shaped one leaf masonry panels, reproducing historical masonry. Tests have been done for evaluating the elastic and shear moduli in both plane directions, with 6 panels rotated by 90 degrees, leading to vertically aligned bed joints, and 3 panels maintained with horizontal bed joints. Compressive tests were executed on 6 masonry panels, 3 of them rotated by 90 degrees. Initial shear strength and shear modulus parallel to bed joints are evaluated through shear tests on 9 masonry triplets. Shear tests are performed on 3 rotated panels, applying an horizontal distributed load, without vertical compression. Attention is paid to the service load state: only the initial phase of the tests is studied. Numerical models are proposed for representing actual masonry behavior, both discrete [1] and continuous [2,3], standard and micropolar, obtained by homogenization procedures [4]. Several numerical analyses are performed for simulating the experimental tests on masonry triplets and panels. The mechanical elastic parameters of both discrete and continuous models are calibrated starting from laboratory data of masonry constituents and then by fitting the results of the initial phases of the experimental tests on masonry specimens.*

1 INTRODUCTION

The assessment of masonry structural behavior is an active field of research, due to the large amount of masonry constructions in Europe and due to the vulnerability of historical masonry buildings in case of seismic actions. On one hand, the evaluation of masonry mechanical parameters by means of laboratory tests is always an important activity, due to the wide range of mechanical and geometrical parameters that may be assumed by the resisting elements and by the connections or joints between them [5]. On the other hand, modeling a composite material like masonry is a crucial task. Discrete models [6] may be adopted for considering separately each masonry constituent and perform analysis at microscale level. Continuous models may be adopted for performing analysis at macroscale level. Their mechanical parameters are often obtained with homogenization approaches by identifying a periodic cell typical of masonry material and by adopting local linear and non-linear mechanical parameters. For this purpose, the standard Cauchy continuum may be adopted by determining homogenized elastic parameters [5,7] or studying nonlinear behavior [8,9,10]. Furthermore, a micrpolar or Cosserat continuum is able to account for an enriched field of displacements and rotations, both for defining elastic parameters [3,4,11,12] and nonlinear behaviour [13,14].

In this contribution, an experimental campaign on masonry material is described, starting from standard experimental tests on small specimens, in order to determine the mechanical behavior of masonry constituent materials, namely bricks and mortar. Then, experimental tests on one leaf square-shaped masonry panels, in compression and shear, are described, together with shear tests on masonry triplets. Attention is paid to the service load state; hence, the elastic behavior of masonry is taken into account, with particular attention to stiffness parameters, that are influenced by material heterogeneity. The first objective of the experimental campaign is to simulate the behavior of historical masonry by means of weak and deformable mortar joints with respect to strong blocks. Current and further developments of this work are and will be dedicated to the material nonlinearity and its numerical representation [15]. Analytical and numerical discrete and continuous models are then introduced for simulating the elastic behavior of the experimental tests on masonry panels and triplets. In particular, the discrete model with rigid blocks and elastic interfaces introduced by one of the authors [1] is adopted, together with Cauchy and micropolar continuum models determined by means of compatible identifications [4]. The mechanical parameters of the numerical models are determined in terms of joint stiffness of the discrete model, and in terms of the components of the elastic tensor for standard and micropolar continua. In the latter case, particular attention is given to the shear components of the elastic tensor and to the different effectiveness of Cauchy and micropolar continua in representing masonry shear behavior.

2 EXPERIMENTAL TESTS

As stated in introduction, a wide experimental campaign was carried out for simulating the behavior of historical masonry. Standard compressive tests on masonry constituents, namely bricks and mortar, were performed for determining their compressive strength and stiffness. Then, compressive tests on one-leaf masonry panels were carried out, together with shear tests on masonry triplets and masonry panels. Geometric and mechanical characteristics of materials and specimens may be found in recent and upcoming contributions proposed by authors and co-workers [15,16]. For instance, Italian standard clay bricks were adopted, having density $\rho = 1800$ kg/m^3 and with the following dimensions: height $a = 0.055$ m, length $b = 0.250$ m, width $s = 0.120$ m. The mortar adopted for layers and joints in masonry panels and specimens was specially produced with a small quantity of hydraulic lime, in order to obtain a material with low stiffness and low strength, for reproducing the characteristics that may be found in a historical material.

2.1 Compression tests on masonry constituents and masonry panels

Compressive tests on bricks were performed on 3 cubic specimens according to UNI EN 772-1 [17], in order to determine their compressive strength, but allowing also to determine the mean value of elastic modulus E_b = 4100 MPa. Compressive tests on mortar were performed on 6 half-specimens, obtained after 3 indirect tensile tests (namely three point bending tests) on mortar prisms, following UNI EN 1015-11 [18]. A very small mean mortar elastic modulus E_m = 200 MPa was obtained, whereas the corresponding mean compressive strength was f_{cm} = 1.13 MPa, that is smaller than that typical of current masonry constructions.

Then, one leaf square-shaped masonry panels were built by arranging the Italian standard clay bricks in a 'running bond' pattern, with 8 blocks along horizontal direction and 16 blocks along vertical direction, and assuming panel thickness t equal to block maximum dimension $t = b$ = 0.25 m,. Horizontal (bed) and vertical (head) mortar joint thickness was assumed equal to $e_v = e_h = e$ = 1 mm, leading to the following panel overall dimensions: length L = 1.03 m, height H = 1.03 m, thickness t = 0.25 m (Fig. 1a,b)

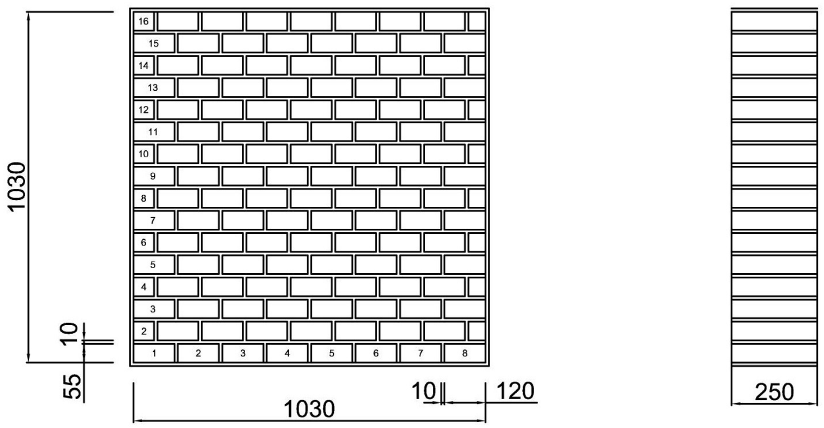

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Figure 1: Front (a) and lateral (b) view of the masonry wall built for the experimental campaign (dimensions in millimeters).

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Figure 2: Compressive tests on masonry panels. Laboratory test configuration for compression normal to bed joints (a) and parallel to bed joints (b).

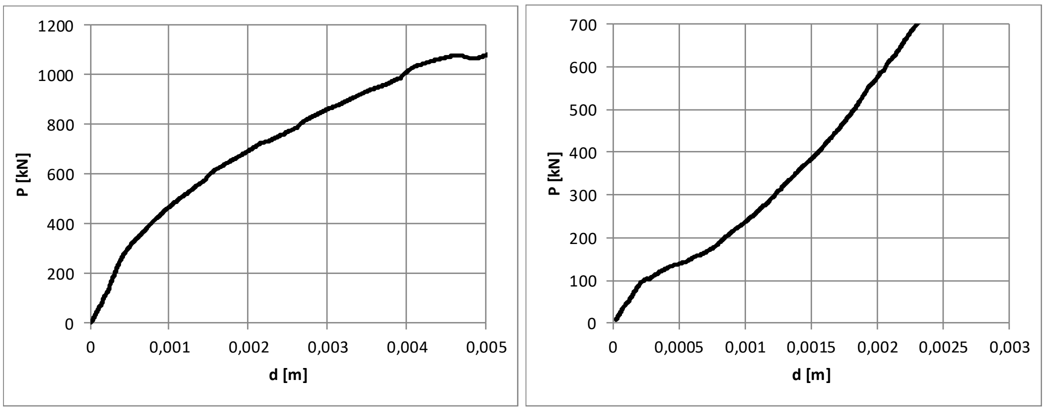

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Figure 3: Compressive tests on masonry panels. Load-vertical displacement curves for compression normal to bed joints (a) and compression parallel to bed joints (b).

Experimental tests were carried out by applying a compressive force both normal (Fig. 2a) and parallel (Fig. 2b) to bed joints. In the latter case, 3 panels were rotated by 90 degrees with respect to the 3 ones of the former case; however, both cases followed UNI EN 1052-1 [19] prescriptions. Fig. 3 shows the mean load-displacement curves for the two panel conditions. In both cases, an initial elastic and stiff behavior is observed, followed by a stiffness reduction of the specimen due to initial damage. In particular, the second loading condition is characterized by a limited range for the elastic behavior (Fig. 2b), followed by a sudden damage characterized by the loss of several external vertical columns of blocks and a consequent stiffness reduction. However, this contribution focuses on the assessment of masonry elastic parameters, hence, more details on the entire experimental campaign, especially accounting for damage and nonlinear behavior may be found in [15] and in further developments of this work.

2.2 Shear tests on triplets and masonry panels

The shear behavior of this type of masonry was evaluated for first by performing simple direct shear tests on 3 small masonry specimens (triplets) made of 3 blocks connected by two mortar joints. According to UNI EN 1052-3 [20], a shear action parallel to mortar joints was applied to the inner block, while the outer blocks were simply supported (Fig. 4a). These tests aimed to evaluate the initial shear strength of mortar joints, namely without compressive forces.

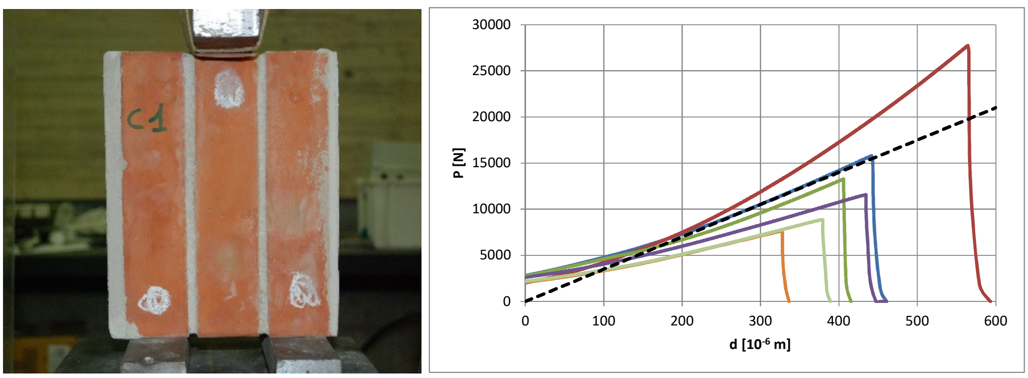

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Figure 4: Shear tests on masonry small specimen (triplet) without precompression. Laboratory test configuration (a) and load-slip curves (b).

Furthermore, results allowed to evaluate the shear stiffness of mortar bed joints. In a recent development of the experimental campaign, the influence of precompression on shear strength has been evaluated, in order to estimate the friction coefficient of the joints [21] and to evaluate the possible influence of precompression on joint stiffness. Fig. 4b shows the load-displacement curves for several experimental tests, together with a linear approximation of the results. Interface mean shear stiffness turns out to be k_s = 0.60 N/mm^3, that is two order of magnitude smaller than that typical of standard mortar joints (for instance k_s = 36 N/mm^3 in [22]).

Then, non-conventional shear tests were performed on 3 square-shaped masonry panels, by applying a horizontal distributed force on the lateral (right) edge of the panel by means of a steel beam along the edge, loaded by an actuator close to the upper (right) corner of the panel. The panel was supported along its base and horizontal displacements were fixed at the base of the opposite (left) edge with respect to the loaded edge. The panel was rotated by 90 degrees, hence with vertically aligned bed joints (Fig. 5). The load and restraint conditions aimed to activate the shear behavior of the panel in the orthogonal direction with respect to head joints, instead of adopting the arrangement of traditional shear tests [23]. Considering the horizontal load-displacement curve, the panel showed an initial elastic behavior up to a horizontal load equal to 20 kN (Fig. 6a), then a first diagonal crack appeared. Furthermore, the relative displacement along the compressed diagonal of the square-shaped specimen was obtained (Fig. 6b) up to the elastic limit.

Figure 5: Shear tests on masonry panels with vertically aligned bed joints, laboratory test configuration.

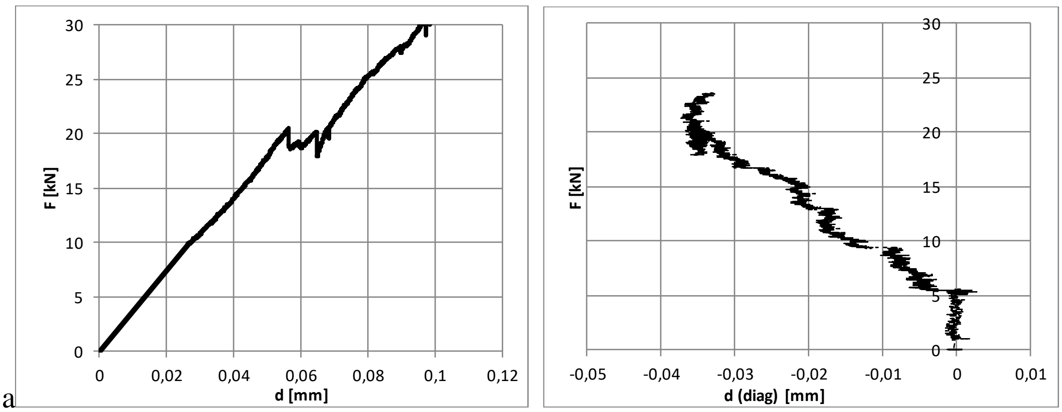

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Figure 6: Shear tests on masonry panels with vertically aligned bed joints. Horizontal load vs horizontal displacement (a); horizontal load vs diagonal relative displacement (b).

3 ANALYTICAL AND NUMERICAL MODELS

As stated in introduction, masonry in-plane behavior is here studied by means of a discrete model with rigid blocks and by means of equivalent continuum models. Numerical tests are performed for simulating the experimental tests on masonry panels and specimens, in order to calibrate the mechanical elastic parameters of both discrete and continuous models by fitting the results of the initial phases of the experimental tests.

3.1 Discrete model

The discrete model introduced in [1] is considered and a Cartesian two-dimensional (2D) coordinate system (y_1 y_2) is adopted, together with plane stress conditions. Brick and mortar stiffness values allow to adopt the hypothesis of rigid blocks, since the ratio between their elastic moduli is E_b/E_m = 20.5. Thanks to this hypothesis, the displacement of each block is defined by the translation of its center and by the rotation with respect to its center $\mathbf{u}^i = \{u_1^i\ u_2^i\ \omega_3^i\}^T$ [1]. Mortar joints are modelled as elastic interfaces by assuming as deformation measure the displacement 'jump' between the adjacent blocks. Interface actions are represented by a normal force, a shear force, and a couple $\mathbf{f} = \{f_n\ f_s\ c\}^T$ and are obtained by integrating normal and shear stresses $\boldsymbol{\sigma} = \{\sigma_n\ \sigma_s\}^T$ over interface area. Assuming a linear elastic constitutive law, $\boldsymbol{\sigma} = \mathbf{Kd}$, interface stresses linearly depend on interface relative displacements $\mathbf{d} = \{d_n\ d_s\ d_r\}^T$, namely relative normal and shear translations and relative rotation, and interface stiffness $\mathbf{K} = \mathrm{diag}\{k_n\ k_s\ k_r\}$, collecting normal, shear and rotational stiffness. These components may be further detailed by distinguishing bed (horizontal) joint stiffness $\{k_n^h\ k_s^h\ k_r^h\}$ and head (vertical) joint stiffness $\{k_n^v\ k_s^v\ k_r^v\}$; however, in the original DEM [1], bed and head joints have the same mechanical parameters, since interface stiffness values depend on Young and shear moduli of mortar:

$$
\begin{aligned}
k_n &= k_r = E_m / (1-\nu_m^2) / e \\
k_s &= G_m / e = E_m / [2(1+\nu_m)] / e
\end{aligned}
\tag{1}
$$

Assuming mortar elastic modulus determined with compressive tests and adopting a standard Poisson's ratio ν_m = 0.2, k_n = 20.8 N/mm^3 and k_s = 8.3 N/mm^3. Interface shear stiffness turns out to be one order of magnitude larger than that determined experimentally, for this reason in the following numerical tests, interface normal and flexural stiffness are calculated starting from mortar elastic modulus, whereas interface shear stiffness is assumed equal to that determined from the shear tests on masonry triplets.

3.2 Equivalent continuum models

A standard Cauchy model or a micropolar model may be defined as 2D equivalent continuum models for the regular masonry considered here [4]. In the Cauchy model, the vector of plane translations **u** collects the kinematic descriptors of the system, and the stress tensor **N** collects in-plane actions, namely normal and shear stresses, that represent the dynamic descriptors of the system. In the micropolar model, the skew tensor $\boldsymbol{\Omega}$ of in-plane rotations ω_3 is assumed as further kinematic descriptor, together with the tensor **M** of in-plane couples as further dynamic descriptor.

A compatible identification between the discrete system and the continuum models allows to obtain the components of the elastic tensors **A** and **L** that characterize constitutive functions for the micropolar continuum [3,4,12]:

$$\mathbf{N} = \mathbf{A}(\operatorname{grad}\mathbf{u} + \boldsymbol{\Omega}),$$
$$\mathbf{C} = \mathbf{M}\,\mathbf{e}_3 = \mathbf{L}\operatorname{grad}\boldsymbol{\Omega}. \tag{2}$$

Whereas for the Cauchy continuum the constitutive function is **N** = **A** sym(grad **u**).

Components of tensors **A** and **L** are written as follows, highlighting interface normal and shear stiffness of horizontal and vertical joints, together with block dimensions of a representative elementary volume (REV) of running bond masonry:

$$A_{1111} = k_n^v b + \frac{1}{4}k_s^h \frac{b^2}{a}, \quad A_{2222} = k_n^h a, \quad A_{1122} = 0, \quad A_{1212} = k_s^h a, \quad A_{2121} = k_s^v b + \frac{1}{4}k_n^h \frac{b^2}{a},$$
$$L_{11} = \frac{b^2}{192}\left[16k_n^v \frac{a^2}{b} + k_n^h \frac{b^2}{a} + 12k_s^h a\right], \quad L_{22} = \frac{b^2}{48}k_n^h a. \tag{3}$$

Whereas, for the Cauchy continuum, $A_{1212}^{\text{Cauchy}} = A_{1212} \cdot A_{2121} / (A_{1212} + A_{2121})$.

4 NUMERICAL TESTS

4.1 DEM results

Fig. 7 shows load-displacement curves of the two different compressive tests simulated by means of the discrete model. Numerical results turn out to depend mainly on interface normal stiffness. Focusing on the first test with compression normal to bed joints (Fig. 7a), the stiffness of the DEM is very close that of the second phase of the test, after the first cracks. Assuming a larger normal stiffness for the bed joints k_n^h, namely by assuming larger mortar elastic modulus E_m = 400 MPa, that may be motivated by the material confinement given by compression, DEM results turn out to fit accurately the initial elastic phase of the test. Focusing then on the test with compression parallel to bed joints (Fig. 7b), DEM turns out to be influenced by both normal stiffness of head joints k_n^v and shear stiffness of head joints k_s^h. Assuming the original value k_n^v = 20.8 N/mm^3, and k_s^v = 0.6 N/mm^3 from triplet tests, DEM results fit the initial phase of the compressive test. If shear stiffness is assumed starting from mortar elastic modulus, the model turns out to be slightly stiffer and quite far from laboratory results.

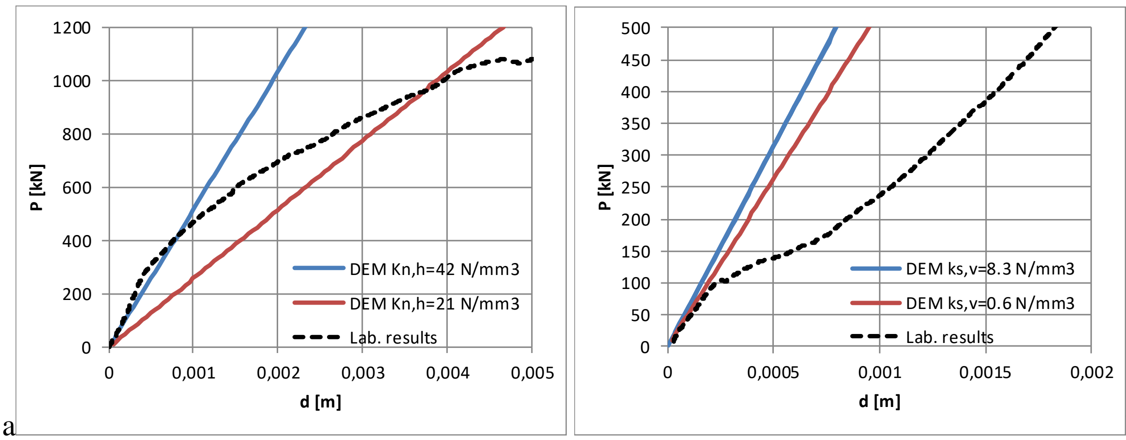

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Figure 7: Compressive tests on masonry panels. Laboratory results and numerical results obtained with DEM and varying several stiffness parameters.

It is worth noting that the discrete model is not able to describe the 'Poisson's effect' given by the deformation of the specimen in the orthogonal direction with respect to the applied load.

The simulation of the shear test on masonry triplet with DEM turns out to be quite simple. The shear stiffness taken from the experiment allows to obtain results coincident with the average laboratory results (Fig. 8). The shear stiffness that can be determined starting from mortar elastic modulus leads to a stiffer model, with displacements smaller than those of the laboratory tests. Similarly, the non-conventional shear test is modelled with DEM adopting two possible shear stiffness values (Fig. 9). In this case, the shear stiffness taken from triplet tests allows to fit the results in terms of both horizontal displacement at the upper-right corner and relative displacement along the compressed diagonal of the panel for almost the entire test, up to the first occurring crack. The shear stiffness determined from mortar elastic modulus allows to fit only the results at the beginning of the test in terms of relative displacement along compressed diagonal of the panel.

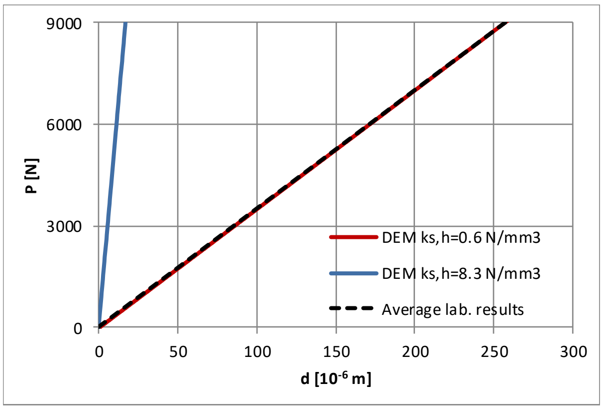

Figure 8: Shear tests on masonry small specimen (triplet). Average laboratory results and numerical results obtained with DEM.

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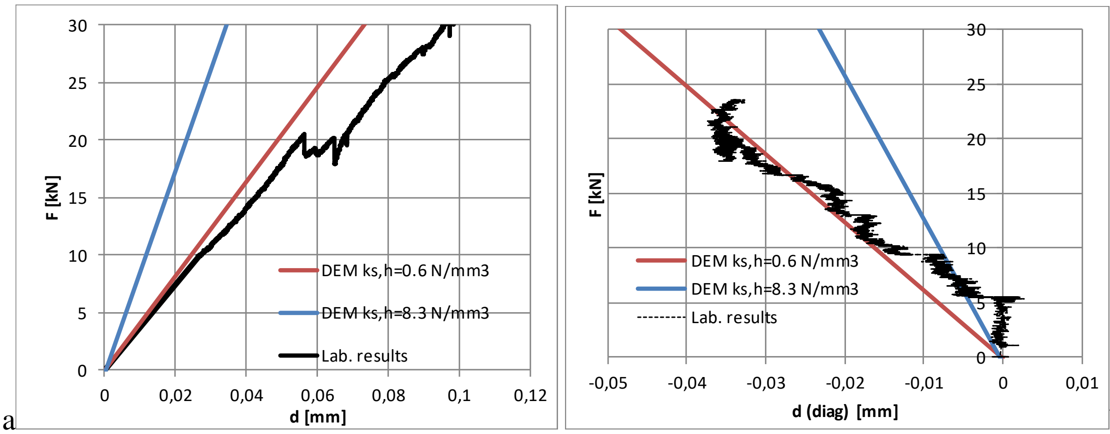

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Figure 9: Shear tests on masonry panels with vertically aligned bed joints. Laboratory results and numerical results obtained with DEM. (a) Horizontal load vs horizontal displacement at the upper-right cornet; (b) Horizontal load vs relative displacement along the compressed diagonal.

4.2 Equivalent continuum FEM results

Numerical tests for simulating compressive laboratory tests are not performed due to their simplicity and given that elastic parameters A_{1111}, A_{2222} are coincident for both continuous models. As well known, differences between DEM and FEM results for a continuous model depend on the number of heterogeneities of the model and, for increasing the number of blocks, DEM and FEM results turn out to converge to the Cauchy solution [4].

Shear tests are more interesting, since they allow to appreciate the different behavior of Cauchy and micropolar models in simulating the shear deformation of a masonry panel; then, the non-conventional shear test is numerically simulated by means of standard and enriched constant stress triangular elements [4] for Cauchy and micropolar continua, respectively. Fig. 10 shows the maps of horizontal displacements over the panel due to a horizontal force equal to 10 kN and applied close to the upper-right corner of the panel obtained with DEM, FEM for Cauchy continuum and FEM for micropolar continuum. The FEM for Cauchy continuum turns out to be more deformable than the other numerical models, whereas the FEM for micropolar continuum is quite close to DEM results and confirms the results obtained in [4].

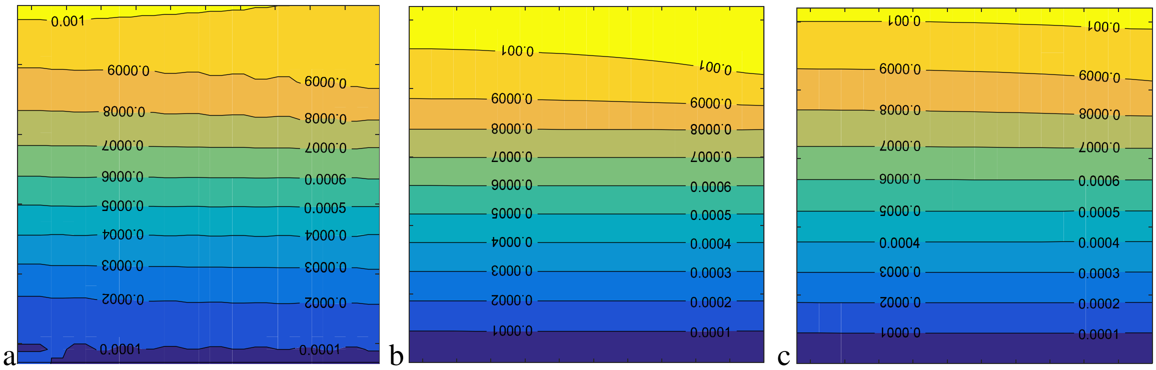

Figure 10: Shear tests on masonry panels with vertically aligned bed joints. Maps of horizontal displacements obtained with DEM (a), FEM for Cauchy continuum (b), FEM for micropolar continuum (c), with a horizontal load at the upper-right corner equal to 10 kN.

5 CONCLUSIONS

- An experimental campaign on specimens and panels reproducing historical masonry was carried out for calibrating mechanical parameters to be adopted in discrete and continuous models. At this stage, only the elastic behavior of masonry and its components is considered.
- A discrete model with rigid blocks and elastic interfaces is taken into account. Stiffness parameters are calibrated from compressive tests on mortar and masonry panels and on shear tests on masonry triplets. Interface shear stiffness turns out to be one order of magnitude smaller than that usually determined from mortar shear elastic modulus.
- Cauchy and micropolar continuum models are also adopted for reproducing masonry behavior. Elastic parameters are determined by means of an identification procedure of the discrete system. The micropolar continuum is able to better account for the masonry texture with respect to the Cauchy continuum. The simulation of shear tests on masonry panels confirms this aspect; the FEM for the micropolar continuum is less deformable than that for the Cauchy continuum and it is closer to DEM results.
- Further developments of this work will consider the nonlinear behavior of masonry and its components and will aim to reproduce the nonlinear behavior and the damage by means of DEM, Cauchy and micropolar models.

ACKNOWLEDGMENTS

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