Laboratory and numerical experimentation for masonry in compression

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Abstract: In this paper, the initial part of a laboratory and numerical experimental campaign dedicated to historical masonry is described. One leaf masonry panels with regular texture are built in order to simulate a historical material characterised by strong resisting elements and weak mortar joints. Laboratory tests are first dedicated to masonry components and then to the behaviour in compression of masonry panels, which is applied both orthogonal and parallel to bed joints, in order to highlight the orthotropic behaviour of the material. First of all, the mechanical parameters of masonry constituents are calibrated and then a heterogeneous finite element model is introduced and calibrated for reproducing the orthotropic behaviour of masonry, together with the initial elastic response and the initial nonlinear behaviour due to the first level of damage.

Keywords: masonry; compression tests; heterogeneous finite element model; model calibration.

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1 Introduction

The assessment of masonry structural behaviour is an active field of research for the community of architects and civil engineers, due to the huge amount of masonry historical buildings that can be found in Europe and Italy in particular. The study of historical masonry behaviour is important both for correctly evaluating the load-bearing capacity of existing structures and for the correct design of strengthening interventions. As well known, masonry is a heterogeneous material composed of resisting elements, namely natural or artificial blocks, connected by dry or mortar joints. Historical masonry is often composed of strong blocks connected by weak and thin mortar joints. Blocks can be arranged more or less regularly, depending on their natural or artificial shape. The evaluation of masonry mechanical parameters by means of laboratory tests is always an important task, 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 (Augenti et al., 2012). This paper focuses on the compressive behaviour of one leaf masonry specimens with regular texture, characterised by strong artificial clay bricks connected by weak lime mortar joints, in order to simulate historical masonry typical of plain territories of northern Italy.

In order to avoid destructive in-situ test campaigns, laboratory tests are generally a good choice for determining masonry mechanical parameters and behaviour under compressive and shear actions. A renewed technologic and scientific interest on masonry behaviour has arisen at the beginning of the last century. Pioneering experimental tests on masonry were carried out by Stang et al. (1928) and McBurney (1928), whereas in the second half of the last century, more and more guidelines and experimental tests were proposed by the scientific community (British Standards Institute, 1936, 1948, 1964, 1970; Graf, 1952; Thomas, 1953; Hilsdorf, 1965, 1969; Monk, 1967; Sahlin, 1971; Page, 1973; Grimm, 1975). Further contributions were added in the last decades (Page, 1981; Sinha and Pedreschi, 1983; McNary and Abrams, 1985; Binda et al., 1988; Vermeltfoort, 1992), together with the first numerical simulations of laboratory tests (Page, 1978; Ali and Page, 1988; Riddington and Naom, 1994) by means of finite element models (FEMs), which were initially based on heterogeneous modelling and were rapidly improved into and compared with continuous modelling strategies (Smith and Carter, 1970; Page et al., 1985) for limiting the computational effort of the analysis (see for further details the review by Tzamtzis and Asteris, 2003). It is worth noting that one of the first effective heterogeneous FEM for masonry (Page, 1978) was characterised by quadrilateral elements for bricks and zero-thickness interface elements for mortar joints. The latter element type was extended from the geomechanics field to the structural masonry one. Similarly, another modelling choice for masonry taken from the geomechanics field is represented by the discrete element model (DEM), originally developed for representing granular materials and rocks fracturing (Cundall, 1971), and effectively extended for representing the discontinuous behaviour of masonry, with particular attention to historical case studies (Lemos, 2007). Even if the time spent for computation is continuously decreasing due to the improvement of computer hardware, heterogeneous FE or DE models can be used for detailed micro-modelling, whereas continuous FE models can be adopted for macro-modelling (Lourenço et al., 1995, 1998).

The research field of masonry testing and numerical modelling is still active (Capozucca, 2004; Lourenço and Pina-Enriques, 2006; Kaushik et al., 2007; Domede and

Sellier, 2008). Recent developments on one hand focus on homogenisation procedures allowing to account for masonry material nonlinearity and complexity by adopting continuous models (Zucchini and Lourenço, 2007; Milani, 2011; Bertolesi et al., 2016), on the other hand, detailed micro-models and numerical tests at micro-structural level are still proposed (Drokugas et al., 2015; Bernat-Maso et al., 2017; Sarhosis and Lemos, 2018), together with discrete models (Baraldi et al., 2018a).

The principal purpose of this paper is the laboratory and numerical assessment of the behaviour of simulated historical masonry panels, composed of standard bricks and weak mortar joints. One of the main aspects of the work is represented by the calibration of a detailed heterogeneous FEM, starting from its constituents, for simulating the actual behaviour of the masonry panels subject to compression both orthogonal and parallel to bed or horizontal mortar joints. Furthermore, these tests allow to highlight the typical orthotropic behaviour of the material both in terms of stiffness and strength.

The paper is organised as follows: the laboratory experimental tests on masonry constituents are presented for first; then, the laboratory compression tests on masonry panels are presented and discussed. These compression tests are simulated by means of a FE model of the panels, and the results of the numerical tests are discussed and compared with the laboratory ones, showing a quite good agreement for both cases of compression orthogonal and parallel to bed joints. The paper ends with several considerations on masonry orthotropic behaviour and on its mechanical parameters, together with the description of the current and future developments of this work, which will take into account the behaviour of masonry subject to shear actions and will consider further numerical models for simulating masonry behaviour, with particular attention to the DEM, the combined finite-discrete element model (FEM-DEM), and standard or micropolar continuum models.

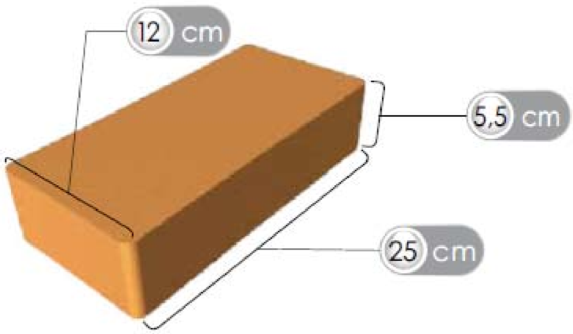
2 Laboratory tests on masonry constituents

Preliminary laboratory tests were performed on masonry constituents, namely bricks and mortar, in order to determine their compressive strength, their elastic moduli, and their stress-strain constitutive laws in compression to be adopted in the FEM.

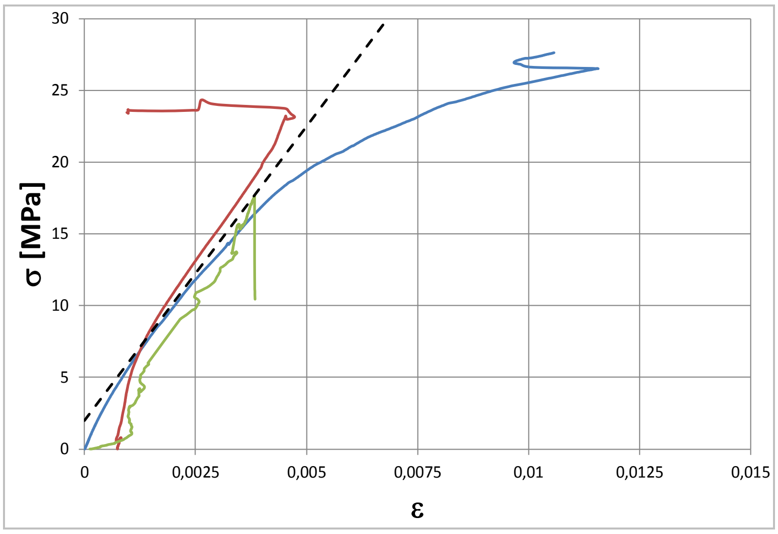
2.1 Compression tests on bricks

Italian standard clay bricks were adopted for the experimental campaign. Brick dimensions are length b = 250 mm, width s = 120 mm, and height a = 55 mm [Figure 1(a)]. Compression tests were performed following most of the guidelines of UNI EN 772-1 (2011), by cutting three cubic specimens from the bricks, in order to determine their compressive strength, but also for determining the mean value of elastic modulus. Average compression strength $f_{c,b}$ and elastic modulus E_b of bricks are collected in Table 1, whereas their mass density, according to producer's technical specifications, is ρ_b = 1,705 kg/m^3. Figure 1(b) shows with continuous lines the stress-strain curves obtained from laboratory tests, whereas the dashed line represents the linear elastic approximation of the tests, obtained adopting the mean elastic modulus.

Figure 1 (a) Clay brick adopted for the experimental campaign (b) Compression tests on brick portions (continuous lines) and estimation of brick elastic modulus (dashed line) (see online version for colours)



(a)



(b)

Table 1 Brick mechanical parameters determined according to UNI EN 772-1 (2011)

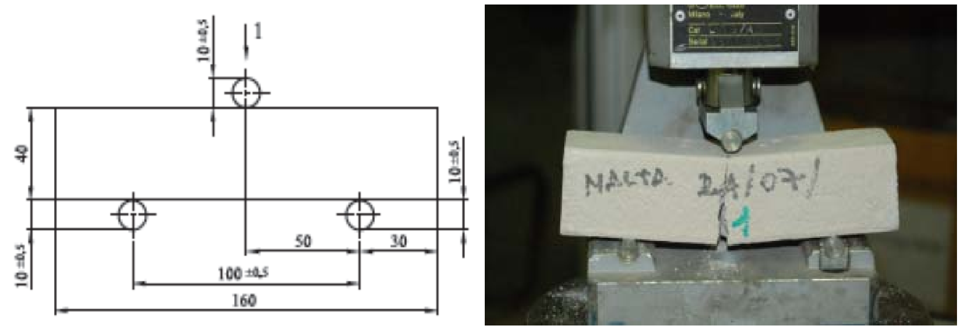
Brick mechanical parameters			
Compressive strength	$f_{c,b}$	22.3	[MPa]
Elastic modulus	E_b	4,100	[MPa]
Mass density	ρ_b	1,705	[kg/m^3]

2.2 Bending and compression tests on mortar

The mortar adopted for bed and head joints in masonry panels 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.

Three prismatic mortar specimens were created and tested following the guidelines of UNI EN 1015-11 (2007), namely having a square section of 40 × 40 mm^2 and length 160 mm. Starting with a three-point bending test on each masonry prism [Figure 2 and continuous lines in Figure 4(b)], the tensile strength $f_{t,m}$ of the material was estimated; then, two compression tests on each half of the original specimen were performed [Figure 3 and continuous lines in Figure 4(a)], allowing to define mortar compression strength $f_{c,m}$. Before performing bending tests, mortar mass density was also found to be ρ_m = 1,623 kg/m^3, which is slightly smaller than the corresponding value of bricks.

Figure 2 Bending test on mortar prismatic specimen, (a) scheme of the test (dimensions in millimetres) (b) damaged specimen at the end of the laboratory test (see online version for colours)

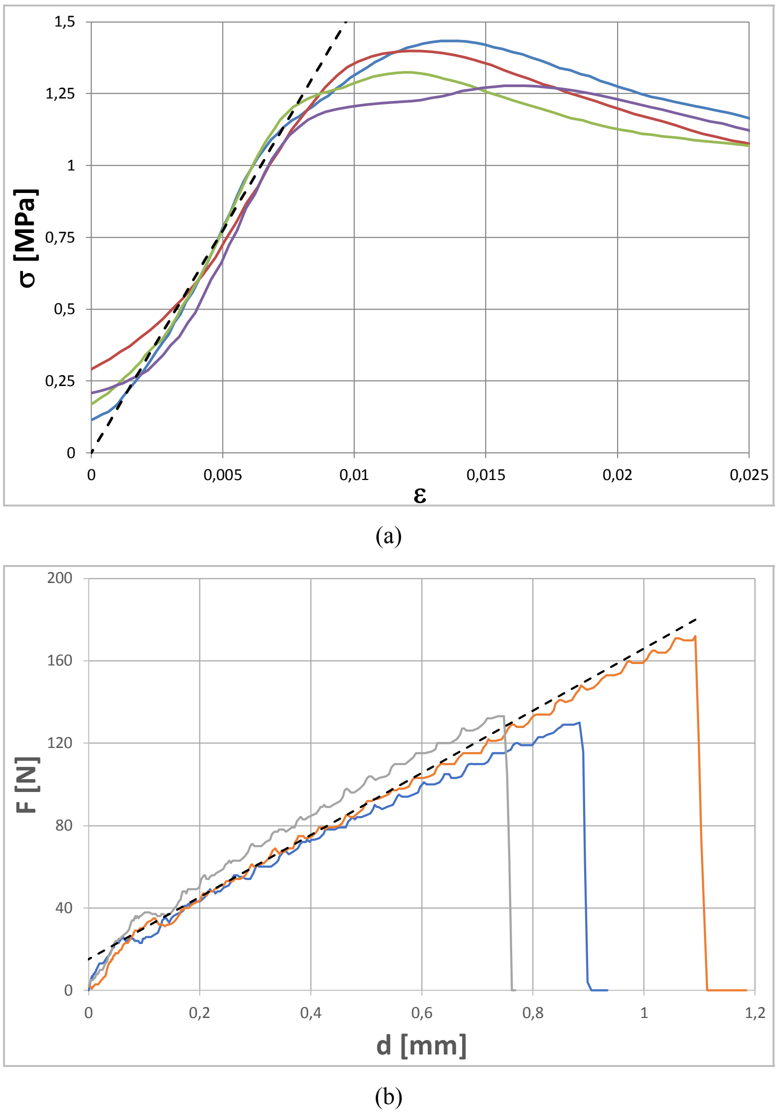


(a) (b)

Figure 3 Compression test on mortar specimen, (a) laboratory test on one half of original specimen (b) two halves of original mortar specimen broken after the compression tests (see online version for colours)

(a) (b)

Figure 4 Laboratory tests on mortar (continuous lines) and estimation of mortar elastic modulus (dashed lines), (a) stress-strain curves of compression tests (b) load-displacement curves of bending tests (see online version for colours)



Mortar elastic modulus E_m was estimated for first by evaluating the initial stiffness of the stress-strain curves obtained from compression tests, see for instance the dashed line in Figure 4(a). Furthermore, mortar elastic modulus was further estimated by considering the slope of the load-displacement curves of bending tests [dashed line in Figure 4(b)]. For this purpose, a Timoshenko beam model was adopted for representing the behaviour of the thick specimen, assuming a standard Poisson's coefficient ν_m = 0.2 and a shear coefficient equal to 1.2. Differently than compression tests, which were characterised by a stiffness reduction for increasing stresses, bending tests turned out to be characterised by an elastic-fragile behaviour, and mortar elastic modulus from bending tests turned out to be one order of magnitude smaller than that determined from compression tests.

The mechanical parameters determined with bending and compression tests on mortar are listed in Table 2.

Table 2 Mechanical parameters of mortar determined according to UNI EN 1015-11 (2007)

Mortar mechanical parameters		
Compressive strength	$f_{c,m}$	1.16 [MPa]
Tensile strength	$f_{t,m}$	0.33 [MPa]
Elastic modulus in compression	E_m	200 [MPa]
Elastic modulus in bending	E_m	210 [MPa]
Mass density	ρ_m	1,623 [kg/m^3]

As expected, mortar compressive strength turned out to be smaller than that typical of contemporary constructions. Furthermore, the corresponding elastic modulus in compression did not follow the common relationships between its value and the compressive strength, given that the elastic modulus of a brittle material is often suggested to be equal to 1000 times its compressive strength.

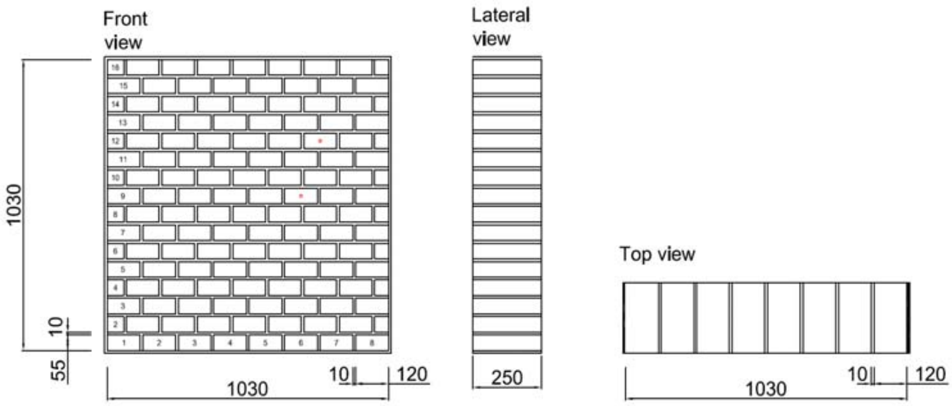
3 Laboratory compression tests on masonry panels

A square-shaped one-leaf masonry panel specimen was defined for the experimental campaign. The specimen is composed by Italian standard bricks previously tested, jointed by bed (horizontal) and head (vertical) mortar layers with equal thickness $e_h = e_v = 0.01$ m. Mortar mechanical parameters have been determined previously, together with mortar joints shear stiffness and strength (De Nardi et al., 2017; Baraldi et al., 2018a), that are not taken into account into this contribution. The overall dimensions of the specimen are (Figure 5): length $L = 1.03$ m, height $H = 1.03$ m and thickness $t = 0.25$ m. Such dimensions are obtained by placing bricks following a 'head-running bond' pattern, with eight bricks in horizontal direction, 16 bricks in vertical direction, and one brick along specimen thickness. This arrangement is chosen on one hand in order to have a smaller size of heterogeneity with respect to panel dimensions, leading to a specimen characterised by 136 resistant elements instead of 72 elements with a standard 'running bond' pattern, which may be stronger against shear actions with respect to the chosen pattern. This aspect has already allowed (Baraldi et al., 2018a) and will allow to better simulate numerically the behaviour of the panels by means of a discrete model and a homogenised standard or micropolar model. On the other hand, the arrangement is chosen in order to avoid panel instability during the laboratory tests, and consequent out-of-plane failure, due to possible eccentricity of the vertical load.

3.1 Compression tests orthogonal to bed joints

Compression tests orthogonal to bed joints were performed on three masonry panels by following most of the guidelines of UNI EN 1052-1 (2001). Figure 6(a) shows the setup of the tests, characterised on each panel side by two vertical transducers along the middle-third of the height, and by one horizontal transducer at panel mid-height. A thick steel plate was placed on the top of the panel for allowing the distribution of the concentrated vertical load, together with two layers of rubber at panel top and bottom edges, in order to avoid the effect of possible surface roughness on load distribution and base restraint effect [Figure 6(b)]. Differently with respect to UNI EN 1052-1 (2001) indications, which envisage a compression test in load control, here, a displacement control was adopted, allowing to observe a softening response of the specimen after reaching the maximum vertical compression.

Figure 5 Front, lateral, and top views of the masonry panel built for the experimental campaign



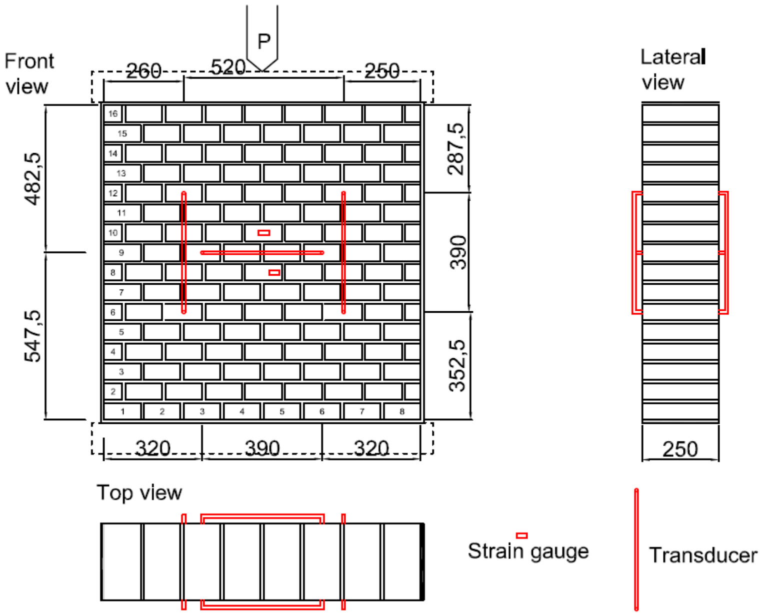
Note: Dimensions in millimetres.

The compression tests were characterised by an initial elastic behaviour up to 1–2 MPa; then, several vertical cracks started to appear on the bricks and head joints close to the vertical edges of the panel. With this damaged condition, panel stiffness decreased, and the compression test continued up to the compressive failure of the specimens, close to 5–6 MPa. Figure 7(a) shows the stress-strain curves obtained by averaging the measurements of the four vertical transducers for two masonry specimens. Measurements from each panel turned out to be in excellent agreement. Assuming the stress-strain curves from vertical transducers as reference, the corresponding stress-strain curves from the load cell at the top of the panel have been scaled, in order to remove the initial elastic deformation of the upper and lower rubber layers. The resulting stress-strain curves [Figure 7(b)] are not able to correctly estimate the initial elastic stiffness of the panel, but the compressive stiffness after the initial vertical cracks is in good agreement with that determined by vertical transducers. Furthermore, the three panels showed a similar behaviour in compression, with only one panel collapsing with a slightly slower value of compressive strength. The average compressive strength for the proposed masonry type is $f_w = 5.19$ MPa.

3.2 *Compression tests parallel to bed joints*

Compression tests orthogonal to bed joints were performed on three masonry panels by extending the guidelines of UNI EN 1052-1 (2001) to a panel rotated by 90° with respect to the previous one, hence, characterised by vertically aligned bed joints. Figure 8 shows the setup of the test, which was carried on in the same manner of the previous one by applying a vertical compression under displacement control.

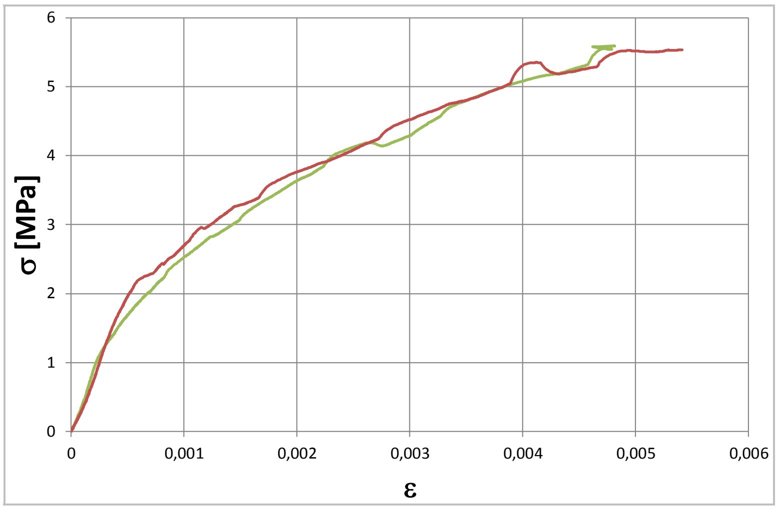
Figure 6 Masonry panel subjected to compression orthogonal to bed joints, (a) front, lateral and top views of test setup (dimensions in millimetres) (b) laboratory test beginning (see online version for colours)



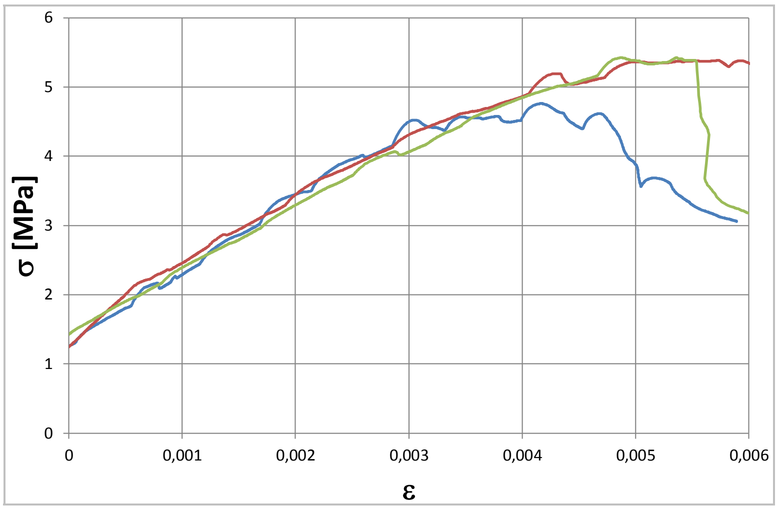
(a)

(b)

Figure 7 Data acquired from compression tests orthogonal to bed joints, (a) average stress-strain curves from vertical transducers (b) stress-strain curves from load cell (see online version for colours)



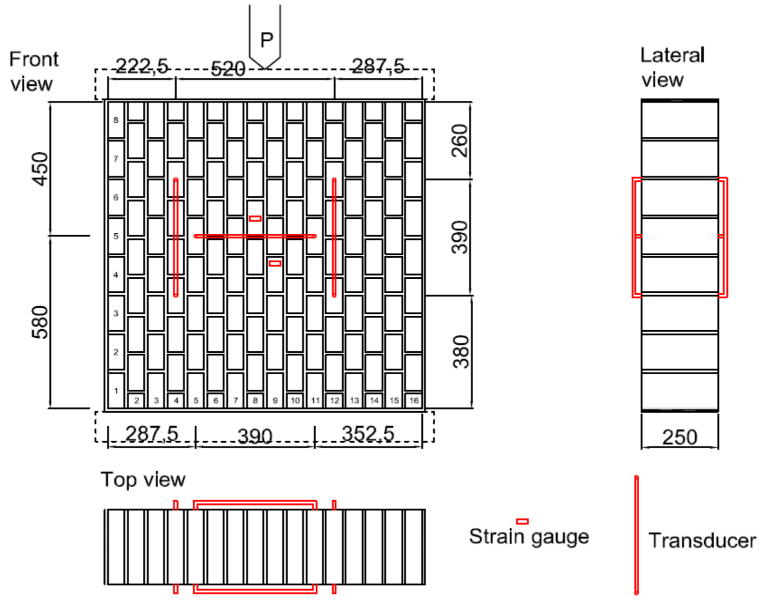
(a)



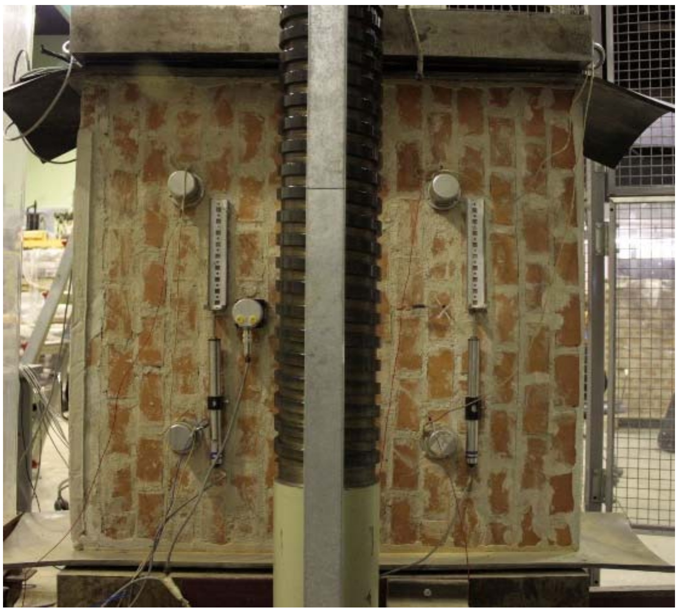
(b)

These compression tests were characterised by a limited initial elastic behaviour, due to the sudden development of cracks aligned with the bed joints close to the vertical edges of the panels. It is worth noting that the transducers gave correct information only with two panels [Figure 9(a)], whereas the third one was characterised by a difficult transfer of the load at the beginning of the test, followed by an extremely fast collapse. The data obtained from the load cell have been scaled by removing the deformations of the upper and lower rubber layers. Despite the initial elastic phase of the test, the slope of stress-strain curves, after the initial cracks, obtained both with the transducers [Figure 9(a)] and with the load cell [Figure 9(b)] turned out to be in very good agreement and the compressive failure was reached at 3–4 MPa, with a fast collapse without softening. The average compressive strength for the proposed masonry type rotated by 90° was f_{w1} = 3.68 MPa.

Figure 8 Masonry panel subjected to compression parallel to bed joints, (a) front, lateral and top views of test setup (dimensions in millimetres) (b) laboratory test beginning (see online version for colours)

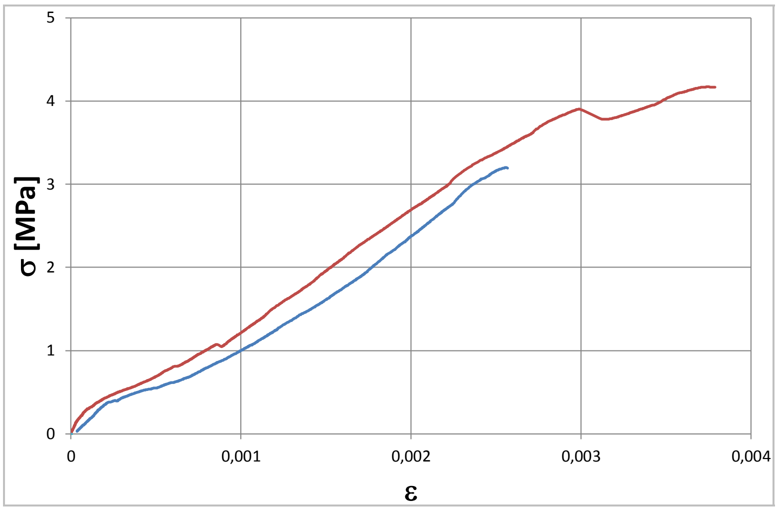


(a)

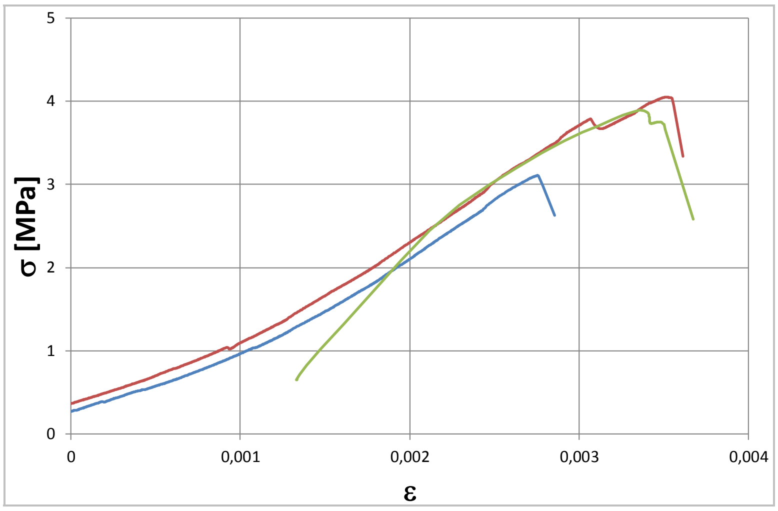


(b)

Figure 9 Data acquired from compression tests parallel to bed joints, (a) average stress-strain curves from vertical transducers (b) stress-strain curves from load cell (see online version for colours)



(a)



(b)

4 Numerical experimentation

A standard heterogeneous FE model was introduced in order to reproduce both compression tests on masonry panels. Bricks and mortar layers were discretised by means of quadrilateral elements with four nodes in plane stress state. Three material types were defined, the first one for brick elements, then two different materials were introduced for bed and head mortar joints, respectively, accounting for a larger strength and stiffness of bed joints with respect to head ones. This latter aspect is justified for first by the good building quality of bed joints, thanks to their horizontal orientation and larger surface. Furthermore, mortar strength and stiffness of bed joints can be considered larger than that of mortar tested separately, thanks to their thickness limited to 10 mm, to the confinement given by the neighbouring bricks, and to the compression applied to the panel. The better performance of bed joints with respect to head joints has been recently underlined and validated in a contribution dedicated to the simulation of the laboratory tests by means of a discrete model (Baraldi et al., 2018a), even if limited to the elastic behaviour.

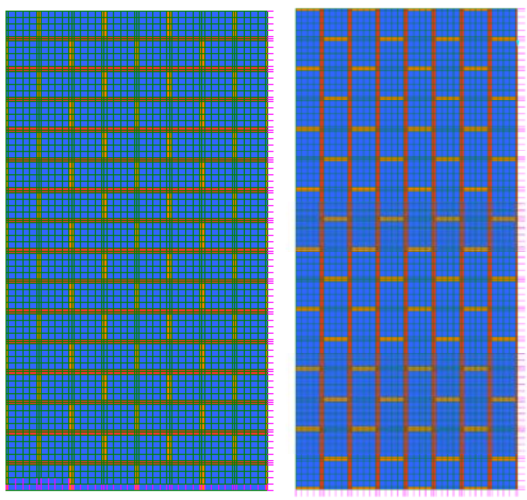
The material nonlinearity was taken into account into the FEM in a simplified manner, by adopting elastic-perfectly plastic stress-strain laws for each material. Starting with bricks, the elastic modulus from laboratory tests was adopted to define the elastic behaviour both in compression and tension, together with the compression strength, whereas an estimated smaller value for tensile strength was assumed, equal to 1 MPa. The parameters for head mortar joints were assumed coincident with those determined from laboratory tests, whereas the parameters for bed mortar joints were assumed two times larger than those determined from laboratory tests.

The mechanical parameters adopted in the FEM are resumed in Table 3.

Table 3 Mechanical parameters of materials adopted for FE analyses

Material	*FEM materials mechanical parameters*				
	Strength [MPa]		*Elastic modulus [MPa]*		*Mass density [kg/m^3]*
	Compression	*Tension*	*Compression*	*Tension*	
	f_c	f_t	E	E	ρ
Bricks	22.30	1.00	4,100	4,100	1,705
Bed mortar joints	22.6	0.66	400	40	1,623
Head mortar joints	1.13	0.33	200	20	1,623

Figure 10 FE model of one half of the masonry panel for simulating the (a) compression orthogonal to bed joints and (b) parallel to bed joints (see online version for colours)

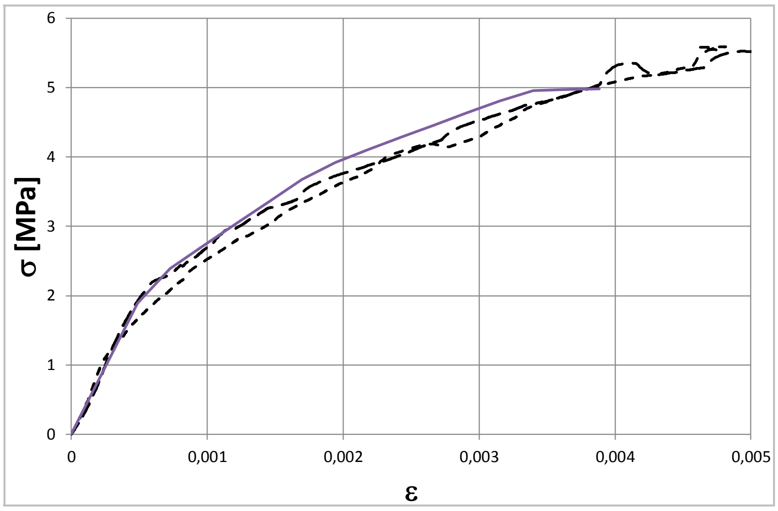


(a) (b)

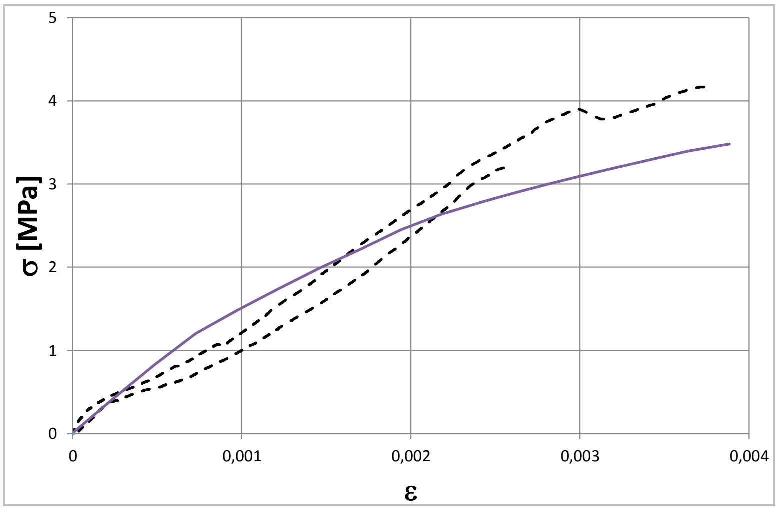
Then, two nonlinear static analyses were performed with the proposed FEM, in order to simulate the two different compressive actions applied to the masonry panels (Figure 10).

Numerical results turned out to be in good agreement in case of compression orthogonal to bed joints [Figure 11(a)], both for simulating the initial elastic behaviour, the first damage, and the subsequent stiffness reduction due to the first level of damage. The ultimate strength reached by the model was close to 5 MPa, which is in quite good agreement with laboratory results.

Figure 11 Stress strain curves for masonry panels subject to (a) compression orthogonal to bed joints and (b) parallel to bed joints (see online version for colours)



(a)



(b)

Note: FEM results with continuous lines, and laboratory results with dashed lines.

In case of compression parallel to bed joints, numerical results turned out to be in sufficient agreement with laboratory results [Figure 11(b)]. In particular, the FEM slightly overestimated the first level of damage and slightly overestimated the subsequent stiffness reduction.

In both cases, the FEM could not be able to correctly represent the damage of masonry panels close to the ultimate strength level, since the actual collapse mechanisms where characterised by the formation of discrete cracks and by the collapse of external portions of the panels. As well known, these types of material nonlinearities cannot be correctly represented by a standard and continuous model, even if characterised by a heterogeneous material.

Final considerations can be done regarding the behaviour of the type of masonry proposed in this contribution. Even if standard and contemporary clay bricks with a high compressive strength level were adopted, the use of a weak mortar specifically created for this experimental campaign allowed to generally reduce the overall compressive strength orthogonal to bed joints.

Among the several suggested expressions for estimating masonry strength (Tassios, 1988), the proposed masonry specimens turn out to be in quite good agreement with the following ones:

$$f_w = \frac{f_{c,b}}{6} + \frac{\sqrt{f_{c,b} + f_{c,m}}}{4} - \frac{f_{c,m}}{20} + 1.4 = 6.34 \text{ MPa} \tag{1}$$

$$f_w = \left(1 - 0.8\sqrt[3]{(1/a)}\right) \cdot \left[f_{c,m} + 0.4\left(f_{c,b} - f_{c,m}\right)\right] = 5.24 \text{ MPa} \tag{2}$$

It is worth noting that the same order of magnitude of strains was reached during the compression tests, and as expected, a smaller compression strength was reached in case of compression parallel to bed joints, with a ratio f_w / f_{w1} = 1.41. Due to the sudden initial damage, the panel subject to compression parallel to bed joints showed a smaller overall stiffness with respect to the standard one compressed orthogonally to bed joints.

5 Conclusions

In this work, a laboratory and numerical experimental campaign dedicated to the assessment of masonry behaviour was described. Attention was given to a particular type of masonry, characterised by strong bricks and weak mortar joints, with the main purpose of simulating a historical material. Standard tests on masonry constituents were performed and deeply investigated in order to determine the mechanical parameters to be adopted in a detailed heterogeneous FEM. Particular attention was given to the orthotropic behaviour, typical of masonry, by performing compression test in orthogonal and parallel direction with respect to bed mortar joints. As expected, masonry panels compressed orthogonally with respect to bed joints showed larger strength and larger overall stiffness with respect to the case rotated by 90°. These standard tests allowed to perform a preliminary calibration of a heterogeneous FEM, and represent a further real case study that can be added to the existing literature of in-plane compression tests on masonry panels.

It is worth noting that the behaviour of bed mortar joints in the numerical model was assumed to be different and stronger with respect to that of head joints that were assumed to follow the parameters of mortar specimens subject to bending and compression. This aspect is often not taken into account when numerical tests are performed independently with respect to real case studies.

As already stated, this work collected a preliminary calibration of the heterogeneous FEM in order to simulate the elastic and initial inelastic behaviour of the panels. For this reason, elastic-perfectly-plastic constitutive laws were adopted. Further developments of this work will regard the simulation of the compression tests presented here, by means of more accurate constitutive laws, for instance accounting for the softening behaviour of masonry constituents. More accurate numerical models will be also adopted, such as the DEM and FEM-DEM already proposed by authors for studying the behaviour of dry masonry subject to horizontal loads (Baraldi et al., 2013, 2018b, 2019). Particular attention will be given to the damage assessment of both compression tests and to the behaviour of the masonry panels subject to shear actions. However, a better simulation of masonry in-plane behaviour should be obtained by keeping a two-dimensional model and by considering the effect of the out-of-plane strains due to the in-plane compressive loading acting on the panel by means of an enriched kinematical model (Addessi and Sacco, 2016a, 2016b, 2019).

It is also worth noting that the tests on masonry constituents and masonry panels here proposed will help the assessment of more complex case study tested both in laboratory and numerically, such as the three-leaves masonry walls with different types of inner core (Aldreghetti et al., 2017, 2018; Boscato et al., 2018a; 2018b).

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