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Experimental assessment of shear strength parameters on rubble stone masonry specimens

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HIGHLIGHTS

- The in plane shear strength of rubble stone masonry walls is studied by means of experimental tests.
- The cohesion of rubble stone masonry panels is assessed by diagonal compression tests.
- The cohesion and the friction coefficient of rubble stone masonry panels are evaluated by triplet tests.
- The obtained experimental results (shear strength parameters) are compared with other experimental works.

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ABSTRACT

Rubble stone masonry walls were widely used in traditional buildings in Mediterranean countries. However, the mechanical behaviour of those walls is not completely characterized due to a lack of experimental data. This paper presents an experimental campaign carried out to characterize the shear strength parameters of traditional Mediterranean rubble stone masonry walls. Fifteen masonry specimens were built using old techniques and traditional materials. Two types of mortar were used in the specimens' execution, air and hydraulic lime mortars, with intend to simulate different masonry construction periods. The goal of the experimental campaign was to evaluate the most important mechanical parameters needed for numerical modelling of traditional rubble stone masonry walls, namely, the compressive strength and Young's modulus through compression tests; the cohesion and friction coefficient by triplet tests; and the diagonal tensile strength via diagonal compression tests. The tests' setup and load–displacement diagrams are presented for all tests and the obtained shear strength parameters are compared with values from the literature.

1. Introduction

Buildings with heritage value require minimal and reversible interventions, which have to be based on surveys and diagnostic studies. A full understanding of the structural behaviour and material characteristics is essential for the restoration and no action should be undertaken without detailed examination of benefits and disadvantages of the adopted methodologies [1].

The mitigation of seismic risk in many Mediterranean urban areas requires the reinforcement of old buildings, which must be preceded by assessment of the actual seismic buildings' safety. For that purpose a complete mechanical characterization of the masonry walls is needed. In the last decades some laboratory and in situ tests have been performed on masonry load-bearing walls, but in literature values for shear parameters on rubble stone masonry walls are still scarce. It must be highlighted the in situ experimental tests performed by Corradi [2–4], Brignola [5,6] and Borri [7,8], which allowed the shear characterization of traditional Italian rubble stone masonry walls by means of diagonal compression tests. However, further tests are needed to obtain supporting data for safety assessment studies of old masonry buildings in other locations. The observation of seismic damage on load-bearing masonry walls showed that masonry panels under in-plane loading may have two typical types of behaviour associated to different failure modes [9,10]: flexural behaviour (rocking with crushing) and shear behaviour (sliding shear failure and diagonal cracking). The occurrence of those failure modes depends on the wall's geometry, boundary conditions, acting axial load and material's characteristics. Different models oriented to describe those specific failure modes are present in the literature [10,11] and codes [12,13]. For diagonal cracking the Turnsek and Cacovic model [14] is usually accepted. In these models the strength domain is defined through a single material parameter: the tensile strength of masonry, usually obtained by diagonal compression tests. For

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sliding, the strength domain is usually defined by two parameters [15,16]: the cohesion and friction coefficients, usually evaluated through triplet tests. However, the experimental evaluation of cohesion and friction coefficients may pose some problems in irregular masonry where the mortar joints are not regularly arranged.

In this paper is described an extensive experimental campaign conducted to characterize the mechanical behaviour of load bearing rubble stone masonry walls existing in old Lisbon buildings. Three types of tests were performed: compression tests, triplet tests and diagonal compression tests. To characterize the compressive strength and Young's modulus, compression tests were conducted on small specimens. In these tests the recommendations of the EN 1052-1 standard [17] were followed. Other prototypes were subjected to triplet tests to quantify the masonry shear strength parameters cohesion and coefficient of friction. Although in rubble stone masonry walls the irregular stone arrangement may prevent the shear sliding, it can happen in constructive (horizontal) joints at the base and at floor levels. In triplet tests the major lines of EN 1052-3 standard [18] and of other works [19–21] were only followed partially, since those publications are related to regular masonry. Diagonal compression tests on masonry specimens were performed to estimate the tensile strength (diagonal cracking failure) of traditional masonry. The test setup and the procedure followed the ASTM E519-02 standard [22] and what is suggested in recent research works [3,5]. The performed tests were intended to evaluate the rubble stone masonry mechanical behaviour and to compare the strength parameters obtained by the different types of tests (cohesion, friction coefficient and tensile strength).

Due to the difficulty of performing all the tests in situ and removing samples from old buildings, masonry specimens were built on the laboratory using traditional materials and techniques: two $40 \times 40 \times 40\,\text{cm}^3$ specimens were built for compression tests; nine $60 \times 40 \times 40\,\text{cm}^3$ specimens were made to perform triplet tests; and four $120 \times 120 \times 70\,\text{cm}^3$ specimens were built to assess their behaviour under diagonal compression. The specimens were built with two different mortars (Table 1): half with air lime mortar (to simulate traditional walls in old buildings) and the other half with hydraulic lime mortar (to simulate walls in less older buildings). The specimens were tested 8 months after their construction to ensure the mortar's hardness.

2. Specimens execution and materials

In order to reproduce the common features of traditional Lisbon construction, the masonry specimens were built with one leaf of roughly cut stones. The stone used in the specimens' manufacture was the most common stone of Lisbon buildings ("Lioz" limestone), which has an average compressive strength of 50 MPa (obtained in compressive tests on 10 cm edge length cubic samples [23]). During the execution the stones were chosen to maximize the fitting and to leave the fewest voids as possible. The biggest stones (with longest edge of about 30 cm) were used in corners and edges and the spaces among them were filled with mortar and small pieces of stone. The mortar was based on two types of binder (traditional air lime and industrial hydraulic lime), which were mixed with a ratio binder/sand of 1/3 (in volume), following the proportions of traditional mortars. The mortars, as well as the masonry specimens, were executed by an elderly stone mason, who was asked to follow traditional techniques. Experimental tests were performed (according to the EN 1015-11 standard [24]) to evaluate the mortars' mechanical properties, such as flexural and compressive strength: nine prismatic mortar specimens ($160 \times 40 \times 40$ mm) were tested for each type of mortar. The obtained mean value for the mortar flexural strength (i.e. tensile strength obtained by bending tests) was 0.35 MPa for hydraulic lime mortar and 0.25 MPa for air lime mortar. Compression tests were also performed on the mortar specimens and the obtained mean values for the compressive strength were 1.47 MPa for the hydraulic and 0.56 MPa for the air lime mortar. Rubble masonry specimens for compression tests were made using stone units with variable shape and dimensions, which were randomly assembled. Specimens for triplet tests were also built with stone units with variable shape and dimensions but special attention was paid in order to create three horizontal layers of stone units (each 14 cm height – Fig. 1) with two almost horizontal bed joints.

The specimens for diagonal compression tests were built in the position where they were tested, (i.e. faces at 45° to the horizontal and vertical) starting from a bottom steel-loading shoe (Fig. 1). Before the test started, a second steel-loading shoe was placed at the upper corner of the specimens, to allow the vertical loading. To study the influence of the stone arrangement in the specimen's tensile strength the stones were applied in horizontal layers in first masonry panel, whereas in the other three panels the stone layers were applied in the diagonal direction, i.e. parallel to the specimens edges. The specimens were built with a 70 cm thickness, which is the typical thickness of the external masonry walls of masonry Lisbon buildings [25,26].

3. Experimental tests

As mentioned, the experimental campaign described in this paper consists of two compression tests, nine triplet tests and four diagonal tests. The tests and the specimens are identified by a two-digit index code in which the first digit indicates the type of test (C: compression test, T: triplet test, W: diagonal compression test) and the second digit is the specimen number. Table 1 lists the specimens and the corresponding tests.

3.1. Compression tests

The aim of the compression tests was to assess the strength and Young's modulus of the masonry under compression. Two masonry specimens (C1 and C2 – hydraulic and air lime mortar specimens, respectively) were compressed by a 3000 kN hydraulic jack. The specimens were centered in the testing machine with one displacement transducer placed on each side of the panels and were loaded continuously till the rupture. The compressive strength f_c and Young's modulus E were evaluated from the experimental data using the following procedure [17]:

$$f_c = \frac{F_{y,\max}}{A} \quad \text{and} \quad E = \frac{F_{y,\max}}{3 \times \varepsilon \times A} \tag{1}$$

where $F_{y,\max}$ is the maximum load reached on a specimen, A is the specimen loaded cross-section, and ε is the strain of the specimen when a load of 1/3 of the maximum load was achieved.

The compression tests results are summarized in Table 2 and depicted in Fig. 2.

The experimental results showed an unexpected similarity between the strength of hydraulic and air lime specimens, which can be explained by the observed failure modes. In both cases the failure involved stone crushing by stone to stone contact and the mortar type had a minor influence on the specimen's ultimate strength. Due to the irregularity of the stone units used in the specimen's execution, the mortar joints had non-uniform thickness with thin layers of mortar between some of the stones' irregularities. As a consequence of the mortar crushing at an advanced stage of loading, some stone edges made contact with each other and so the stones' strength governed the specimens' behaviour under compression. The failure by stone crushing explains the similitude between the strength of the two specimens and, also, the unusually high compression strength obtained for both types of masonry. This effect would probably be diluted in a larger specimen, where it would be felt a larger influence of the mortar quality. On the other hand, in the compression tests reported in this paper, the lateral expansion at the top and bottom of the specimens was restricted by the loading plates, resulting in a confinement effect that tends to increase the capacity of the specimens. Due to the dimensions of the specimens, the in-plane confinement effect that may exist in long walls was not simulated. Due to these differences the results of the compression tests must be considered essentially indicative.

Table 1
Masonry specimens.

Test	Hydraulic mortar	Air lime mortar	Dimensions (cm)
Compression	C1	C2	$40 \times 40 \times 40$
Triplet	T1, T2, T3, T4, T5	T6, T7, T8, T9	$60 \times 40 \times 40$
Diagonal compression	W1, W4	W2, W3	$120 \times 120 \times 70$

Fig. 1. View of a triplet test specimen with three horizontal layers (left) and of diagonal compression test specimens (right).

Table 2
Compression tests results.

Masonry Specimens	Type of mortar	$F_{y,max}$ (kN)	f_c (MPa)	E (GPa)
C1	Hydraulic lime	1282	8.01	1.64
C2	Air lime	1186	7.41	0.56

E is the secant value at 1/3 of the ultimate load.

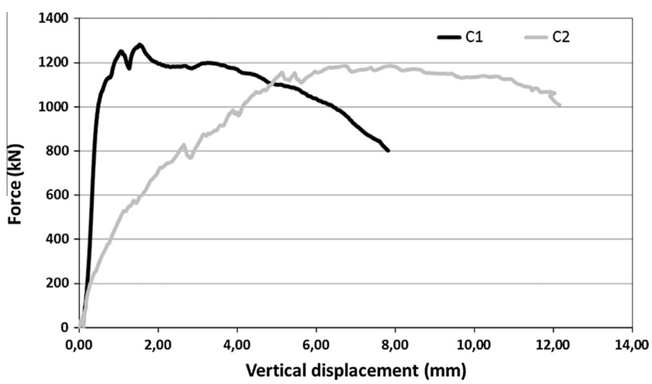


Fig. 2. Compression tests – "force–displacement" diagrams.

3.2. Triplet tests

To quantify the shear strength parameters of horizontal bed joints in rubble stone masonry, triplet tests were performed on nine specimens. The rubble stone masonry specimens were subdivided into two groups, depending on the type of mortar used in its execution: hydraulic lime mortar specimens (T1–T5) and air lime mortar specimens (T6–T9). Following EN 1052-3 [18], the specimens were submitted to a vertical compression load and then a horizontal load was applied at the middle stone layer. The horizontal movement of the upper and lower stone layers was prevented by rigid supports (Fig. 3) and the horizontal load was gradually increased till the sliding of the middle stone layer. The vertical load's magnitude was defined by the actual state of stresses of load bearing walls found in old Lisbon masonry buildings. Three different vertical stress levels were adopted for air lime specimens (Table 3): 0.1 MPa, 0.3 MPa and 0.5 MPa, which were kept constant as much as possible during the test. Since the collapse load is directly dependent of the vertical stress, which is due to the overhang structure and does not depend on the type of masonry, the ratio of vertical stresses to the masonry's resistance tends to be smaller for hydraulic lime panels, where higher strength was expected. Therefore, the 0.5 MPa compression level was not considered in the hydraulic lime specimens to avoid extremely high horizontal forces on the setup. For specimens with hydraulic lime mortar, vertical stresses of 0.1 MPa, 0.2 MPa and 0.3 MPa were applied (Table 3).

The displacements were recorded by thirteen linear voltage displacement transducers (LVDTs) placed on the four specimens faces (Fig. 4). On the front and back faces six transducers measured horizontal displacements and two transducers measured vertical displacements. On the face where the horizontal load was applied one transducer was placed at the actuator and two horizontal transducers measured the middle layer horizontal displacement. On the opposite face two other horizontal transducers were placed.

According to EN 1052-3 [18] the specimen shear strength f_v is obtained from:

$$f_v = \frac{F_{h,\max}}{2 \times A} \tag{2}$$

where $F_{h,\max}$ is the maximum horizontal force (shear load) and A is the cross-sectional area of the two shear surfaces. For moderate compressive stresses the shear strength of mortar joints in regular masonry can be given by a Mohr–Coulomb formulation [27–30], which can also be assumed for the present case. Therefore, the shear strength f_v of horizontal bed joints in rubble stone masonry specimens submitted to a compressive stress σ is given by:

$$f_v = f_{vo} + \mu \times \sigma \tag{3}$$

where μ and f_{vo} stand for coefficient of friction and cohesion, respectively.

As stated in EN 1052-3 [18], the parameters $f_{vo}$and µ can be obtained from several triplet tests performed with different compressive stress levels by means of linear regression.

In triplet tests, as expected, all specimens collapsed by sliding of the middle stone layer (Fig. 5) and higher shear strengths were obtained for higher compression levels. The results are summarized in Table 3 and the "force–displacement" diagrams are depicted in Figs. 6 and 7. Transducer TSH2 recorded the horizontal displacement plotted in the "force–displacement" diagrams and the load cell placed next to the horizontal jack measured the force magnitude. The points where the linear elastic behaviour ends and the points of maximum horizontal force are also marked in the "force–displacement" diagrams.

Comparing with other triplet tests, test T6 showed some peculiarities in the specimen's behaviour. The force–displacement diagram shows a relatively long plateau with slight hardening, registering the maximum horizontal force at relatively large horizontal displacement. This hardening behaviour may be attributed to a stronger interlocking effect of the stones along the nearly horizontal failure surface.

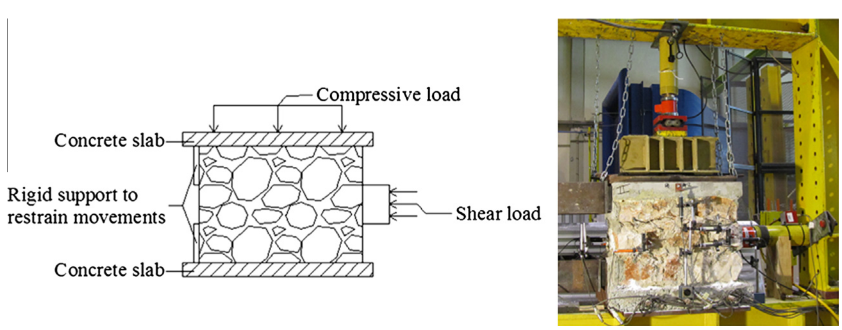


Fig. 3. Setup for triplet test.

Table 3
Triplet tests results.

Specimen	Type of mortar	Vertical force (kN)	Vertical stress σ (MPa)	Maximum horizontal force (kN)	Shear strength f_v (MPa)	Shear strength f_v (average value) (MPa)
T1	Hydraulic	24	0.1	126	0.26	0.33
T2				188	0.39	
T5		48	0.2	213	0.44	0.44
T3		72	0.3	267	0.56	0.57
T4				279	0.58	
T6	Air lime	24	0.1	64	0.13	0.13
T7				56	0.12	
T8		72	0.3	139	0.29	0.29
T9		120	0.5	161	0.34	0.34

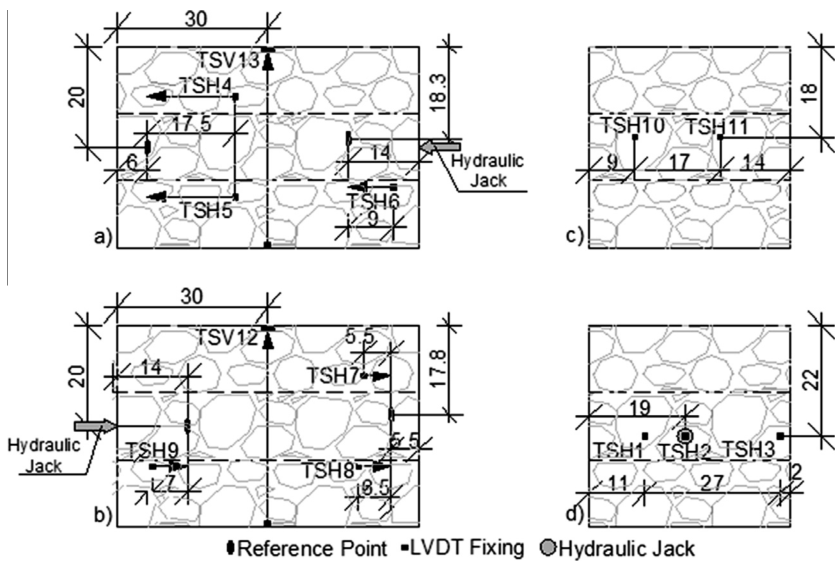


Fig. 4. Triplet tests – transducers on the specimen's front (a) and back (b) faces, on the restrained lateral face (c) and on the loaded lateral face (d) (dimensions in (cm)).

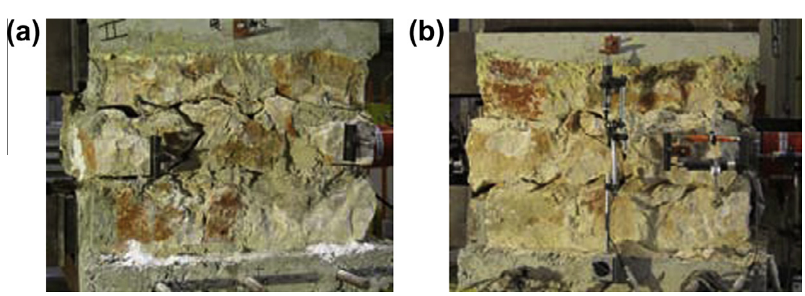
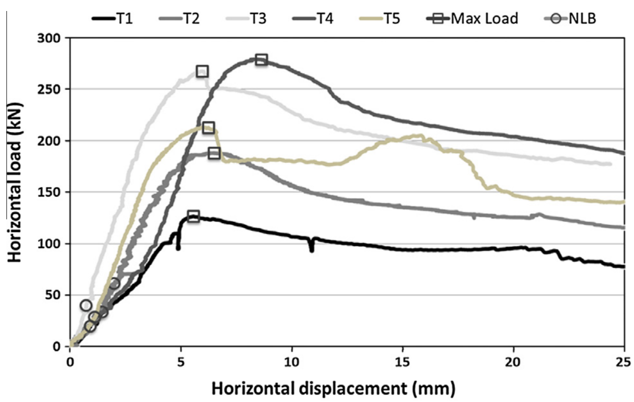


Fig. 5. Crack pattern of masonry specimens: (a) specimen T1 and (b) specimen T7.

As observed in test T7 (Fig. 7), without this hardening effect the maximum load would be slightly smaller and would have been registered at a much smaller horizontal displacement.

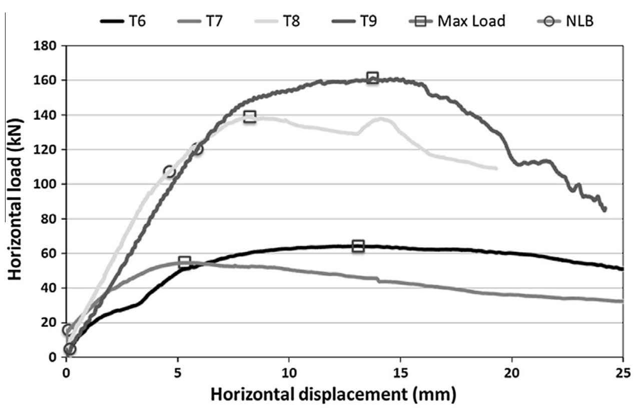
An important issue regarding shear tests is the dilatancy of masonry joints. The dilatancy (i.e. the relation between the vertical and the horizontal strains) has a significant role in numerical modelling and can be much more relevant in rubble stone masonry walls than in brick masonry walls. This is because after cracking and sliding the two sides of the cracks do not match, indicating an increase in volume. This effect increases with the irregularity of the crack surfaces, therefore it tends to be stronger in rubble stone masonry than in brick masonry. Figs. 8 and 9 show the vertical displacement (that can be considered as measuring dilatancy) as a function of the horizontal displacement. It is important to note that the vertical displacement showed is the average of displacements recorded by LVDT's 12 and 13 and the corresponding horizontal displacement was recorded using the transducer placed on the horizontal actuator (TSH2). In Figs. 8 and 9 the points where the linear behaviour ends and the points of maximum force are also marked. It can be noticed that the vertical displacements started to increase after the end of the linear behaviour. The small slope of the "vertical vs. horizontal displacements" diagram around the point of maximum load detected at test T6 (Fig. 9) can be explained by the occurrence of the maximum force for a large horizontal displacement.

Fig. 10 depicts the relation between vertical load and horizontal displacement for all tests, where it can be seen that during the tests the vertical load was almost constant, as required.



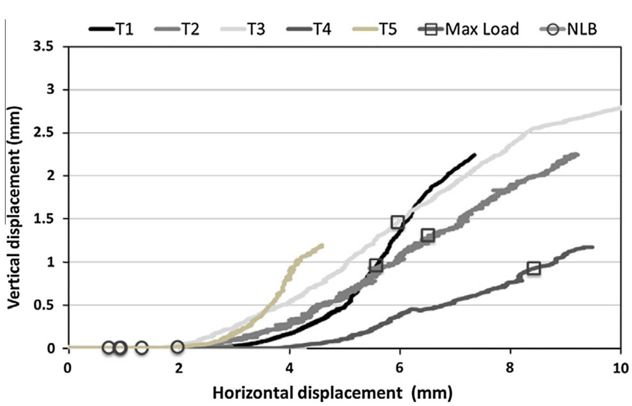
(Max Load – point of maximum load; NLB – point of ending of linear behaviour).

Fig. 6. Triplet tests – "force–displacement" diagrams for hydraulic lime mortar specimens.



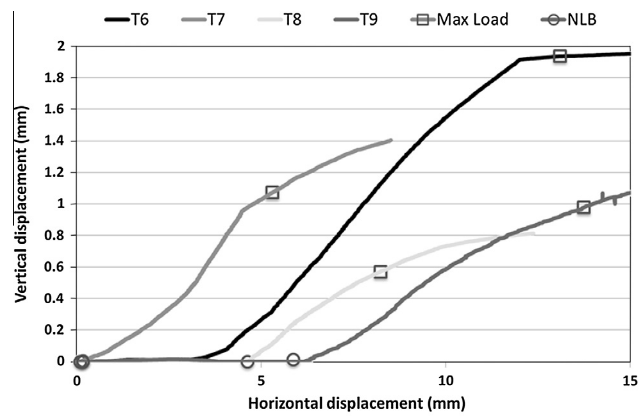
(Max Load – point of maximum load; NLB – point of ending of linear behaviour).

Fig. 7. Triplet tests – "force–displacement" diagrams for air lime mortar specimens.



(Max Load – point of maximum load; NLB – point of ending of linear behaviour).

Fig. 8. Triplet tests – vertical vs. horizontal displacements for hydraulic lime mortar specimens.



(Max Load – point of maximum load; NLB – point of ending of linear behaviour).

Fig. 9. Triplet tests – vertical vs. horizontal displacements for air lime mortar specimens.

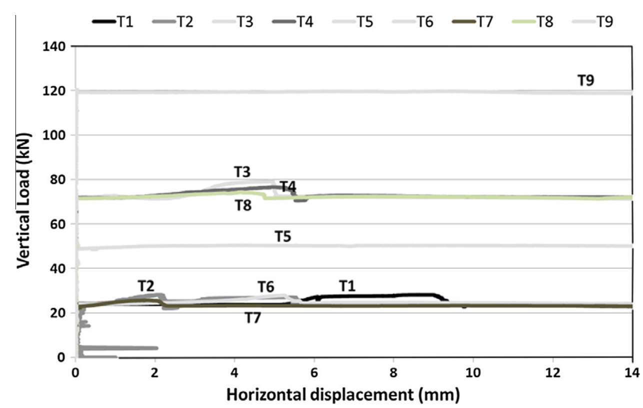


Fig. 10. Triplet tests – vertical load vs. horizontal displacement.

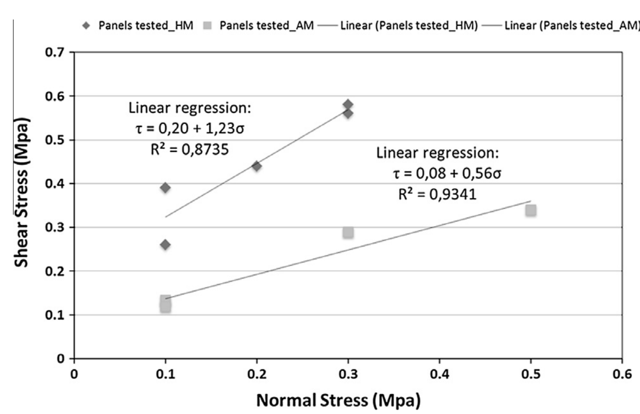


Fig. 11. Triplet tests – relation between shear stress and normal stress for hydraulic and air lime mortar specimens (*R* is correlation coefficient of the linear regression).

As stated in EN 1052-3 [18], the linear regression of results from several triplet tests performed with different compressive stress levels provides the shear strength parameters of the Coulomb's friction model, namely the initial shear strength f_{vo} (cohesion) and the coefficient of friction μ. Fig. 11 shows the relation between the vertical compression stress and the shear strength for all tests. Two straight lines, one for each type of mortar specimen, evaluated by linear regression are also presented in the graph. It is worth mentioning the good correlation between the experimental results and the linear regression lines, which confirms the initial assumption of Coulomb's friction law for the shear strength of horizontal bed joints in rubble stone masonry specimens. For hydraulic lime mortar specimens the values obtained by linear regression for cohesion and coefficient of friction were 0.20 MPa and 1.23, respectively. For air lime mortar the obtained values were 0.08 MPa for cohesion and 0.56 for coefficient of friction.

It must be mentioned that the two shear strength values obtained for the hydraulic lime masonry panels, tested with the lower vertical stress level (0.1 MPa), are quite different (Δ = 33%) and a third triplet test with this vertical stress would have increased the accuracy of the calculations. The low shear strength value obtained in test T1 (0.26 MPa) that may induce a high value for the calculated hydraulic lime specimen's friction coefficient, must be confirmed.

Table 4 summarizes some results published in the literature obtained by triplet tests on regular masonry (regular units with horizontal bed joints). Different values for shear strength parameters were obtained for different types of masonry. For specimens with hydraulic lime mortar and stone bricks the cohesion and the friction coefficient values are around 0.3 MPa and 0.7, respectively. Even though that the tests on rubble and regular stone masonry are not directly comparable, the previous values give some confidence to the obtained results for rubble stone masonry.

Table 4
Triplet tests – literature review.

Masonry type	Cohesion f_{vo} (MPa)	Coefficient of friction μ	
Tuff units, pozzolanic mortar	0.3	2.0	Prota, et al. [19]
Hollow units, cement mortar	1.6	0.9	Gabor, et al. [31]
Solid brick units, cement mortar	0.7	0.7	Amadio and Rajgeli [32]
Solid brick units, hydraulic lime mortar	0.2	0.8	Magenes and Calvi [33]
Sand stone units, hydraulic lime mortar	0.3	0.7	Binda, et al. [34]
Bricks, hydraulic lime mortar	0.2	0.6	Roberti, et al. [35]

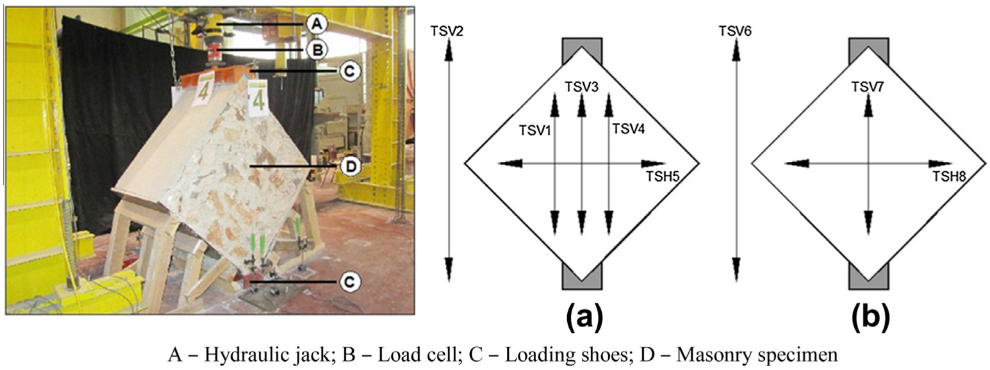


Fig. 12. Setup for diagonal compression tests.

3.3. Diagonal compression tests

In the diagonal compression tests the masonry specimens were placed in the test rig with a diagonal axis in the vertical direction (Fig. 12) and loaded in compression along this direction (causing a diagonal tension failure with the specimen splitting apart along the vertical diagonal). The vertical load was applied by an 800 kN hydraulic jack acting on the steel-loading shoe placed at the top specimens corner.

Nine linear displacement transducers were placed on each specimen (Fig. 12): five transducers on the front face, three on the back face and one to measure vertical displacement under the hydraulic jack. To avoid damages on the instrumentation, the transducers were removed (except the one placed on the hydraulic jack) when the behaviour of the specimen indicated that it could be close to failure. After that the vertical displacement was continuously increased at the top of the specimens until the collapse.

The analysis and interpretation of the diagonal compression tests differ from author to author and from standard to standard. The test was introduced to simulate a pure shear stress state and the ASTM standard [22] assumes that the diagonal compression test produces a uniform shear stress and a Mohr's circle centered in the origin of the Cartesian system of axis. In that case the tensile strength of masonry f_t is equal to the shear cohesion f_{vo} and is obtained by:

$$f_t = f_{vo} = \frac{0.707 \times P}{A_n} \tag{4}$$

where P is the maximum load applied by the vertical jack and A_n is the net area of the specimen, calculated as follows:

$$A_n = \left(\frac{w+h}{2}\right) \times t \times n \tag{5}$$

where w is the specimen width, h is the specimen height, t is the thickness of the specimen and n is the percentage of the unit's gross area that is solid, expressed as a decimal (in the present work the value $n = 1$ was adopted).

The RILEM code [36] considers that the stress field is not uniform and proposes the following expression to evaluate the tensile strength of masonry:

$$f_t = \frac{0.5 \times P}{A_n} \tag{6}$$

Both interpretations are able to represent the experimental crack pattern, located along the diagonal, at 45°.

Brignola et al. [5] proposed a new and more accurate methodology for the evaluation of the tensile strength of masonry from diagonal compression tests with the following expression:

$$f_t = \frac{\alpha \times P}{A_n} \tag{7}$$

where α is a coefficient dependent on masonry typology. For irregular masonry α should be lower than 0.5 ($\alpha = 0.35$ for rubble stone masonry) and for regular masonry α can be assumed equal to 0.5, coherently with the RILEM proposal.

The shear elastic modulus G is obtained by:

$$G = \frac{\tau_{1/3}}{\gamma_{1/3}} \tag{8}$$

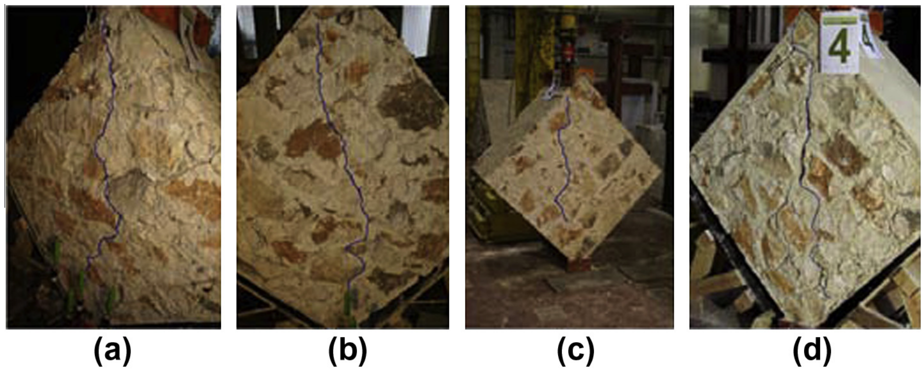
where $\tau_{1/3}$ is the shear stress for a load of 1/3 of the maximum load P_{max} and $\gamma_{1/3}$ is the corresponding shear strain.

As mentioned, four specimens were tested under diagonal compression – two built with hydraulic lime mortar (W1 and W4) and two with air lime mortar (W2 and W3). In all tests the specimens' collapse was caused by a main crack developed in the middle of the specimen, continuously propagating from the center towards the upper and bottom specimen's corners. It should be mentioned that in all tests the stones were not damaged and the crack appeared only through the mortar, dividing the specimen in two almost symmetrical parts (Fig. 13).

Despite of the collapse quasi-brittle nature in all cases, the specimens showed different behaviour after the collapse: the specimens with air lime mortar (W2 and W3) disintegrated, while the specimens with hydraulic mortar (W1 and W4) broke in two parts, each remaining in one piece. The "force–displacement" diagrams (where the vertical displacement is the average value of the measurements recorded using LVDTs 3 and 7), for hydraulic lime and air lime mortar specimens are presented in Figs. 14 and 15, respectively. The results are also summarized in Table 5. The dotted parts of the curves in Figs. 14 and 15 were obtained using the measurements of the transducers under the hydraulic jack, instead of the average values for all measurements. Some unloading and loading branches are visible on the "force–displacement" diagrams, which were due to pauses during which the applied load decreased as a consequence of the damages development. The pauses could not be avoided, as it was necessary to stop the loading to analyse the crack patterns or to remove transducers.

For specimen W1 the maximum applied load was 372 kN with a vertical shortening of 1.55 mm (Point 1 – Fig. 14). The collapse occurred at a larger vertical displacement (5.29 mm) with a load of 268 kN (Point 2 – Fig. 14). In the case of the specimen W4 the collapse occurred when the maximum load was applied: 306 kN with a vertical displacement of 3.47 mm (Point 3 – Fig. 14). The more (apparent) ductile behaviour of specimen W1 is related to the stone arrangement, since the W1 specimen was built with horizontal stone layers (at 45° to the external inclined surfaces), whereas, to be representative of real masonry walls, the other three specimens were built with diagonal layers (at 45°). As expected, the specimens built with air lime mortar showed much lower strength than the specimens based on hydraulic lime mortar. The ultimate load for specimen W2 was 29 kN, with a vertical shortening of 1.58 mm (Point 1 – Fig. 15), and for the specimen W3 the ultimate load was 28 kN, with a vertical displacement of 1.52 mm (Point 2 – Fig. 15).

In the diagonal compression tests the specimens' collapse was achieved without damage to the stones, i.e., the cracks propagated through the mortar joints. That fact is in accordance with the observed major influence of the mortar type in the specimen's strength. In addition, the values for tensile strength obtained by diagonal compression tests (Table 5) are much lower than the values obtained for the mortar's tensile strength by bending tests (0.35 and 0.25 MPa for hydraulic and



a) specimen W1; b) specimen W2; c) specimen W3; d) specimen W4.

Fig. 13. Diagonal compression tests – main crack at the middle of the specimens.

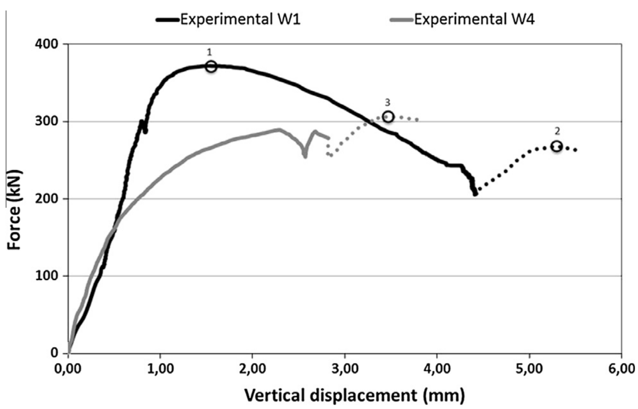


Fig. 14. Diagonal compression tests – ''force–displacement'' diagrams for hydraulic lime mortar specimens.

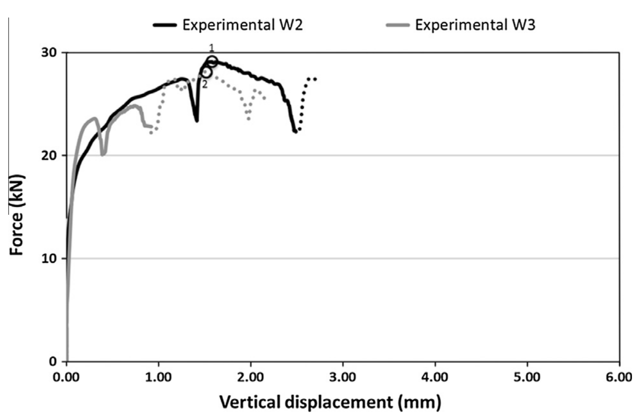


Fig. 15. Diagonal compression tests – ''force–displacement'' diagrams for air lime mortar specimens.

air lime mortars, respectively) and the relative differences between the tensile strength of hydraulic and air lime mortar specimens is much more marked in the diagonal compression tests. This can be due to the fact that the phenomena that conditions rupture is different in both cases, i.e.: in the flexural test it is the tensile strength of the mortar and in the case of the diagonal compression test it is the tensile resistance of the interface between the mortar and the stones.

Moreover, from the differences in tests W1 and W4, whose specimens were built with different stone arrangements, and from the similarities obtained in tests W2 and W3, specimens of which were built with the same stone arrangement, it can also be concluded that the stone arrangement influences (by a moderate degree) the masonry strength and its deformation capacity.

For the air lime mortar specimens (W2 and W3) the maximum compression loads were similar but the shear elastic modulus G varies significantly. The reported variation can be due to the fact that the shear modulus is evaluated on the undamaged stage, with small displacements, where measurement errors may have an important influence. For specimens built with hydraulic lime mortar (W1 and W4) the variation of the shear modulus results can also be explained by the different stone arrangement adopted on the specimens.

Table 6 summarizes some published results obtained by in situ diagonal compression tests on traditional masonry and in Table 7 are shown the values of tensile strength and Young's modulus proposed by the Italian Standard NTC08, 2008 [13] for traditional masonry. Comparing the literature values (Tables 6 and 7) with the

Table 5
Diagonal compression tests.

Masonry specimen	P_{max} (kN)	ASTM		RILEM	Brignola et al. [5] α = 0.35
		f_t (MPa)	G (MPa)	f_t (MPa)	f_t (MPa)
W1	372	0.313	389	0.220	0.155
W2	29	0.024	58	0.017	0.012
W3	28	0.024	93	0.017	0.012
W4	306	0.258	252	0.182	0.128

Table 6
Diagonal compression (in situ) tests – literature review.

Traditional Masonry (lime mortar and calcareous stone)	$f_t = \alpha \times p/A_n$		
	α	f_t (MPa)	
Double-leaf roughly cut stone masonry	0.707	0.05–0.07	Corradi [3]
Random rubble stone masonry	0.707	0.06–0.16	Chiostrini [37]
Random rubble stone masonry	0.5	0.04–0.11	Chiostrini [37]
Random rubble stone masonry	0.35	0.02–0.04	Brignola [5]
Double-leaf roughly cut stone masonry	0.707	0.02–0.04	Borri [8]
Triple-leaf roughly cut stone masonry	0.707	0.02–0.04	Borri [8]
Random rubble stone masonry	0.5	0.04–0.07	Brignola [6]

Table 7
Reference values of the mechanical parameters (maxima and minima) NTC08, 2008 [13]f_t = average tensile strength; E = average value of the elastic modulus; G = average value of the shear modulus.

Traditional Masonry (lime mortar and calcareous stone)	f_t (MPa)	E (MPa)	G (MPa)
Irregular stone masonry (pebbles, erratic and irregular stone)	0.03–0.05	690–1050	230–350
Uncut stone masonry with facing walls of limited thickness and infill core	0.05–0.08	1020–1440	340–480
Cut stone masonry with good bonding	0.08–0.11	1500–1980	500–660

obtained values in the performed diagonal compression tests it can be concluded that the obtained results deserve some confidence, even though that the number of tested specimens of each type of mortar was limited.

4. Conclusions

The goal of the present research programme was to assess the most important mechanical parameters of specimens similar to traditional load bearing walls on rubble stone masonry. Fifteen masonry specimens were specially built for the present experimental program using traditional techniques and materials (lime stones, air lime and hydraulic lime mortars). Samples made with hydraulic lime mortar were built to simulate the latter stage of traditional structural masonry construction in Portugal.

The experimental program discussed in this paper focused on the three types of tests available to characterize the masonry mechanical behaviour: compression, triplet and diagonal compression tests.

Compression tests allowed the evaluation of compressive strength and the Young's modulus for traditional stone masonry and triplet tests were used to estimate the masonry cohesion and coefficient of friction. The obtained results on compressive tests (f_c = 7.41 MPa; E = 0.56 GPa for air lime mortar specimens and f_c = 8.01 MPa; E = 1.64 GPa for hydraulic lime mortar specimens) must be regarded as indicative values due to the limited number of performed tests and to the test boundary conditions. Two contradictory effects of the test boundary conditions may have affected the results, i.e.: the dimensions of the specimens do not reproduce the in-plan confinement of real long loadbearing walls; and the confinement imposed by the bearing plates at the top and bottom of the specimens does not exist in real situations.

Typical failure modes were observed on the triplet tests and the obtained results for cohesion and coefficient of friction were, respectively, 0.08 MPa and 0.56, for air lime mortar specimens and 0.20 MPa and 1.23, for hydraulic lime mortar specimens. The high value obtained for the friction coefficient of hydraulic lime mortar specimens could have been a result of the low strength value obtained in test T1. However, the coefficient of friction on those specimens should have been significantly higher than that obtained for air lime mortar specimens.

To obtain the tensile strength that bounds the strength domain of the (diagonal cracking) Turnsek and Cacovic model [14], diagonal compression tests on four specimens were also performed. Two specimens were built with hydraulic mortar and the other two with air lime mortar. The tested specimens showed brittle behaviour with low values for tensile strength, especially in the case of the air lime mortar specimens (f_t = 0.024 MPa, according to ASTM [22] methodology). The specimen built with hydraulic mortar and stone layers parallel to the faces reached a medium value for the initial shear strength of about f_t = 0.258 - MPa (according to ASTM [22] methodology). It was noted in the shear tests that the mortar composition has an important influence on shear strength. The tests also showed that the stone arrangement could influence the shear strength, as the specimen with layers at 45° to the faces showed shear strength 21% above a similar specimen with stone layers parallel to the faces (both with hydraulic mortar). It must be said that the representativeness of the obtained results depends on the limited number of specimens tested on diagonal compression. However, they are inside the range of values obtained by other researchers for similar masonry specimens.

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