Experimental evaluation of the shear strength of aerial lime mortar brickwork by standard tests on triplets and non-standard tests on core samples

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Abstract – This paper presents the results of an experimental program carried out in the laboratory to evaluate the shear strength of aerial lime mortar brickwork. Masonry triplets and walls were tested after one year from their construction by adopting two different testing methods. The first approach consisted in the shear tests of masonry triplets, whereas the second technique was based on core drilling from walls of 90 mm diameter cylindrical specimens to be subjected to Brazilian tests with varying inclination of the diametric mortar joint. The first method is more adequate to characterize new masonry, whilst the second one is a suitable MDT procedure for the analysis of existing structures. The experimental results from standard and non-standard tests were properly investigated in order to obtain the shear failure envelope of the bond interface and mortar joint. The comparisons between the different tests and their interpretative theories show the possibilities of the novel non-standard testing method for the evaluation of the shear strength of structures of the built cultural heritage.

Keywords: Masonry; Triplets; Coring; Double Punch Test; Brazilian Test; Minor Destructive Testing (MDT); Mohr-Coulomb Criterion; Friction; Cohesion; Bond Interface.

Abbreviations - MDT: Minor Destructive Testing; LVDT: Linear Variable Differential Transformer; DPT: Double Punch Test.

1. Introduction

The mechanical characterisation of existing masonry structures faces significant difficulties due to the intrinsic complexity of masonry as a composite material, the large heterogeneity normally associated to masonry construction and the dependence of the quality of masonry with the construction practice. An adequate characterisation is also hindered by the insufficient standardization available for the mechanical testing of existing masonry materials and structural members.

Nevertheless, a sufficient characterisation of the main resisting parameters is essential for a reliable structural verification of masonry structures. The determination of the compressive strength of existing brickwork is necessary to assess the current safety level of structures of the built heritage subjected to gravitational loads and, more specifically, to future loads caused by changes of use. In turn, the investigation of the shear strength is of paramount importance to evaluate the potential capacity of existing structures against horizontal loads induced by wind or earthquake. The recent earthquakes experienced in the Mediterranean basin, and especially in Italy, have motivated an increasing research effort on the validation of experimental procedures for the investigation of the strength capacity of existing masonry.

Solid clay bricks and lime mortar joints constitutes a typical material combination in historical masonry structures. However, only a limited number of comprehensive experimental programs is available in the literature about the mechanical characterisation of pure lime mortars in the laboratory [1,2]. One of the main reasons explaining such limited available experience may be found in the difficulties related to the long curing and hardening time required by these materials and especially by aerial lime mortar. Recent experimental researches were carried out in the laboratory using pure lime mortar brickwork in order to evaluate its compressive behaviour [3,4]. Masonry built with mixed cement-lime mortar has deserved more significant attention [5–7] than that prepared with lime mortar. More specifically, the contributions available in the literature on the laboratory characterisation of the shear behaviour of pure lime mortar brickwork [8,9] are definitely more scarce than those focusing on masonry with lime-cement mortar or cement mortar [10,11]. In these works, shear tests on triplets or couplets were mainly investigated in order to obtain a shear failure envelope for the joints and the bond interface. The triplet test is established by the EN 1052-3:2002 [12] and is suitable for the shear characterisation of new masonry. The application of the triplet test to historical structures is not possible due to the impossibility found in extracting this type of sample from existing brickwork.

The shear behaviour of existing masonry can be directly evaluated by the in-situ diagonal compression test according to ASTM 509-2010 [13] and RILEM LUM B6 [14]. Several contributions are available in the literature about this experimental technique [15,16] that was also compared with the triplet test in the laboratory [17]. However, this type of experiment is destructive and can be hardly applied to historical buildings. Recent construction standards like the Italian ones [18,19] offer a practical alternative by providing some pre-calculated reference values that may be used by professionals depending on the type of constituent materials and the level of knowledge acquired about the specific case of study through inspection activities.

The extraction of sufficiently large samples from existing structures, allowing the application of conventional laboratory mechanical tests, is hampered by the intrinsic geometrical characteristics of

the masonry texture, the fragility of the investigated material and the difficulty in bringing to the laboratory intact specimens after the sampling and transportation operations. In the case of historical structures belonging to the built cultural heritage, additional difficulties emerge due the need to minimize the damage caused by the inspection. In this case, the investigation of the mechanical properties must be done in a respectful way so that the damage is limited to an indispensable minimum. This allows the inspected structure to be easily repaired whereas the cost of the minor signs due the inspection is largely surpassed by the benefit provided by the allowed gain of knowledge about the material properties. Following this approach, Minor Destructive Testing (MDT) seems an interesting possibility nowadays, since it is based on in-situ sampling of only small specimens that can be tested in the laboratory afterwards. Among the MDT techniques aimed at the assessment of the shear strength of historical masonry, an approach experiencing rising interest is the extraction of small core samples (typically with 70÷110 mm diameter) to be subjected to Brazilian tests with variable inclinations of the diametric mortar joint interposed between two cylindrical segments of brick. This method was proposed firstly by considering a fixed 45º inclination of the diametric mortar joint during the Brazilian test [20,21]. This preliminary approach was aimed at obtaining an empirical correlation between the results from Brazilian tests on core samples and diagonal compression tests on walls.

Recent works proposed a novel comprehensive methodology combining different Brazilian tests with varying inclinations of the diametric mortar joint [22–24]. This method was applied to single leaf walls with half brick thickness, by obtaining core samples with length equal to the thickness of the wall, and also to single leaf walls with one brick thickness. In this second case, as well as in the case of thicker structural members (e.g. two or three bricks thick), the long cylinders extracted from the whole thickness of the wall may present internal vertical joints and thus their transversal cutting is necessary in order to obtain the suitable specimens. In this case, the cut can provide adequate specimens composed of two solid brick cylindrical segments and one diametric mortar joint. The referred researches investigated the behaviour of masonry made of clay bricks and hydraulic lime mortar with low strength, i.e. a frequent material combination in historical materials. The Brazilian tests on core samples showed the shear failure of mortar for specimens with 45º to 60º inclination of the diametric mortar joint.

Other researches [20,25] focused on masonry with lime-cement or cement mortar characterised by rather high strengths and thus the results could be considered more representative of masonry of more recent construction. In particular, Brazilian tests on core samples with very strong cement mortar exhibited splitting of the brick and thus the approach was not suitable to provide further evaluations of the shear strength of the mortar.

This paper constitutes a novel contribution focusing on the shear behaviour of aerial lime mortar brickwork. Despite the aforementioned difficulties in building and testing this type of material in the laboratory, the experimental program presented herein considered masonry specimens tested one year after their construction. The research presents a comparative study considering both standard shear tests on triplets and non-standard tests on core samples. The triplets were built in the laboratory, as well as masonry walls replicating historical aerial lime mortar brickwork. In the second case, the cylindrical samples were extracted by simulating in the laboratory the in-situ core drilling of an existing structural member. The methodology proposed in recent works by the authors [22–24] is followed since it can efficiently combine the results from Brazilian tests on cylindrical specimens with variable inclination of the diametric mortar joint and the results from double punch test (DPT) of mortar joints extracted from the brickwork. This approach combines the results from different experimental tests into the same interpretative theory. The experimental results on the shear behaviour obtained using the non-standard procedure is compared with the outcomes from standard triplet tests with the same age and kind of aerial lime mortar brickwork. The study provides direct relationships between standard and non-standard tests that may result helpful in works dealing with the material characterisation of structures of the historical heritage built with low-strength masonry.

2. Experimental program

The experimental campaign was carried out at the Laboratory of Technology of Structures and Construction Materials of the Technical University of Catalonia (UPC-BarcelonaTech). This section presents information about the material components, the construction of the specimens and the testing setups.

2.1 Material components

This experimental program considered material components intended to reproduce one of the most frequent material combinations in historical masonry, i.e. solid clay bricks and lime mortar joints. Handmade terracotta bricks and pure aerial lime mortar CL90 (without cement) [26] were used for this purpose. The mechanical properties of the material components were assessed on prismatic specimens in compliance with the available technical standards.

According to EN 1015-11:2007 [2], prismatic samples with dimensions 160 × 40 × 40 mm^3 were prepared to evaluate the strength of CL90 mortar. Flexural strength f_{fm} was evaluated on four prismatic specimens (Figure 1a), whereas the compressive strength f_{cm} was assessed on the eight halves produced

by the splitting of the prisms from the four flexure tests. Those halves were tested in compression under 40 × 40 mm^2 steel platens (Figure 1c). Both flexure and compression tests were carried out using a load cell of 10 kN in load control, but with different loading rates of 10 N/s and 50 N/s, respectively. Table 1 summarizes the experimental results that were obtained after one year from mortar pouring. The average values are f_{fm} = 0.55 MPa and f_{cm} = 1.63 MPa and their coefficients of variation (CV) are 9.5% and 5.3%. The average values of flexural and compressive strengths fall within the typical ranges of these properties for historical lime mortar.

The nominal dimensions of the terracotta bricks were 305 × 145 × 45 mm^3, even though the effective dimensions of each unit could slightly vary due to the handmade type of construction. Six units were tested under compression according to the EN 772-1:2011 [27], using a load cell of 3000 kN. The average compressive strength was $f_{cb,u}$ = 30.7 MPa with CV = 5.6%. Besides the standard compression tests on whole units, compression tests on prismatic samples cut from the bricks were carried out by following the EN 1015-11:2007 for mortar [2]. Sixteen brick prisms, with dimensions 160 × 40 × 40 mm^3, provided an average flexural strength $f_{fb,x}$ = 3.63 MPa (CV = 3.7%) along the stretcher direction and $f_{fb,y}$ = 3.79 MPa (CV = 13.1%) along the header direction (Figure 1b). Six brick cubes of about 35 × 35 × 35 mm^3 (Figure 1d) provided an average compressive strength $f_{cb,c}$ = 18.4 MPa with CV = 5.9% in the vertical direction. The cubic samples provided a mean compressive strength lower than that obtained from the whole units. This result was due to the different shapes of the two tested specimens, since the brick units had aspect ratios much higher than those of the brick cubes. As a consequence, the units presented a larger surface in contact with the steel loading platens of the compression machine. This fact generated a higher horizontal confinement effect due to the contact friction. The more pronounced triaxial compressive stress state inside the units produced higher values of the maximum vertical compression force than in cubic samples. Table 2 summarizes the experimental results that were obtained from brick samples.

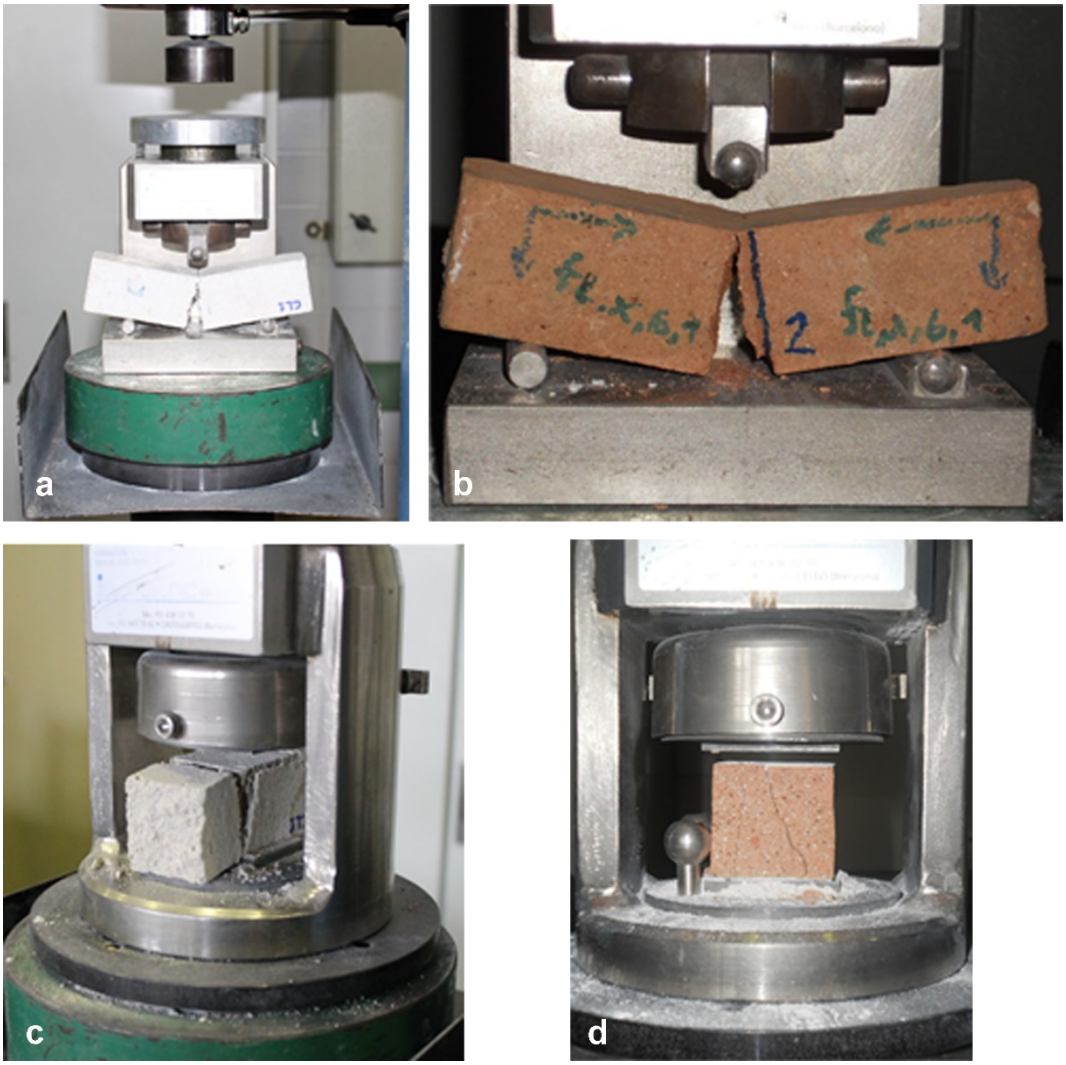

Figure 1 – Testing of prismatic samples of material components: flexure and compression tests on mortar (a,c) and brick (b,d).

2.2 Construction of masonry specimens

The materials described in the previous section were employed to build two walls in stretcher bond with dimensions 1605 × 870 × 145 mm^3 (Figure 2a). The two walls reproduced portions of structural members that may be encountered in typical historical buildings made of low-strength masonry. They were used in this research to simulate, under laboratory controlled conditions, a generic in-situ sampling activity consisting in the core-drilling of 90 mm diameter cylindrical specimens. Each wall was built in-between two steel C beams that were placed below the base and over the top of the specimen. The beams were interconnected by four low-tensioned tie rods. In this way, a very small pre-compression was applied to the walls with the sole purpose of guaranteeing their easy transportation inside the laboratory, as well as their sufficient stability during the coring operations.

At the same time of the construction of the two walls, nine triplets were also built using the same materials, having dimensions 305 × 145 × 165 mm^3 and including three units and two mortar joints (Figure 2b).

Figure 2 – Prismatic specimens of masonry built in the laboratory: a) walls and b) triplets.

After one year from the construction, the two walls were taken out of their storage to carry out the extraction of the cylindrical samples (Figure 3a). The core drilling followed a dry procedure that was proposed in previous works by the authors [4,24]. The absence of water is a remarkable advantage of this improved sampling technique, since it minimizes the risk of disjointing the cylindrical specimens. The wet core drilling, commonly adopted for concrete structures, is not fully adequate for existing masonry buildings since the water may wash the mortar joints away and spoil the samples. On the other hand, the dry core drilling uses an aspirator to cool the bit and to remove the dust. This important feature allows the optimization of the sampling procedure by increasing the number of successful extractions of cores. This is very important in the study of historical constructions, where the minimization of the impact of the inspection on the existing structure is mandatory. As soon as they were extracted, the masonry cylinders were properly confined by strong adhesive tape stuck on the lateral curved surface to avoid any damage during their subsequent storing and transportation. Figure 3b shows the wall after the core-drilling. As it can be noticed, the cut of the bit is precise and does not damage the surrounding material. The procedure proves to be remarkably clean thanks to the absence of dust that is completely drawn by the vacuum cleaner conveniently connected to the drilling

equipment. This feature of the technique, together with the total absence of water, makes the dry core drilling a suitable sampling method, even for the extraction from the interior of existing buildings. Masonry core samples with 90 mm diameter were extracted for this research (Figure 3c). They included a diametric mortar joint interposed between two cylindrical segments of brick. Additional 150 mm diameter cores were extracted to be used in another experimental study. Thirty-one core samples with one diametric mortar joint were extracted from the two walls. Although some of them disjoined during the core drilling, fifteen undamaged samples could be successfully assigned to the Brazilian tests.

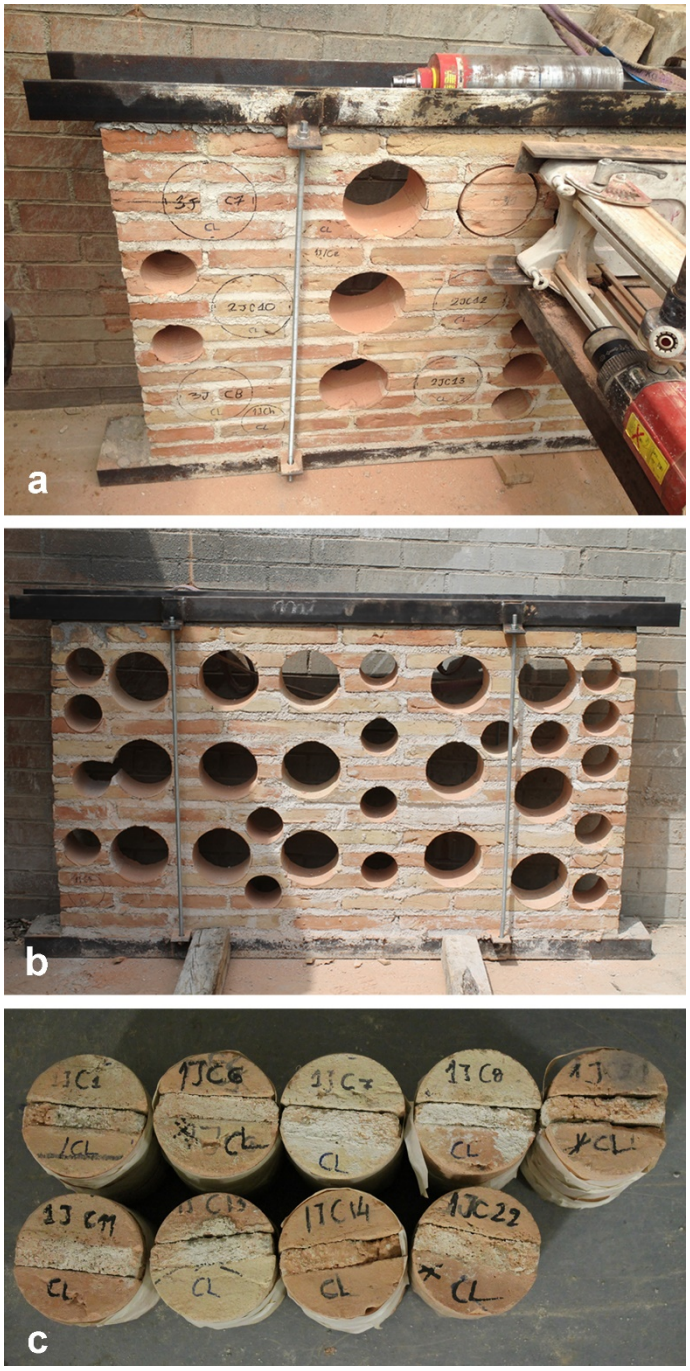

Figure 3 – Extraction of the cylindrical specimens from masonry walls: a) core drilling, b) wall at the end of the extraction and c) 90 mm diameter cylinders for Brazilian tests.

2.3 Testing setups

Both core samples and triplets were tested one year after their construction [28]. Nine triplets were considered first to determine the shear strength of horizontal bed joints in a conventional manner according to the standard EN 1052-3:2002 [12]. All the specimens were tested in a machine with a load cell of 200 kN to apply the shear force to the centre brick. The experimental setup is shown in Figure 4a. The force normal to the bed joints was applied by a 10 kN loading jack. Three triplets were tested for each level of normal stress, i.e. 0.3 MPa, 0.6 MPa and 1.0 MPa. Each normal stress level was kept constant during the test, whilst the increasing shear force was applied to the mortar joints of the triplet. Four Linear Variable Differential Transformers (LVDTs) with ±5 mm range and 5 μm precision were used to control the tangential displacements in the mortar joints during the shear tests. Fifteen core samples were subjected to Brazilian tests with variable inclinations of the diametric mortar joint. Due to the novelty of this kind of test, there are not specific standards. The only standard related, to some extent, with the tests carried out is the EN 12390-6:2000 [29] about the splitting test of concrete specimens. Hence, the experimental setup was the same adopted in previous works by the authors [22–24]. The core samples were laid horizontally between the loading platens. Two wooden stripes were inserted between the platens and the specimen, in order to distribute correctly the diametric longitudinal load. The specimen was compressed along two opposite longitudinal generators until failure. The test was carried out under displacement control at a rate of 0.005 mm/s. The core samples were tested for different inclinations α of the diametric mortar joint with respect to the horizontal direction, as shown in Figure 4b. This peculiar experimental setup allows the application of a combined compression-shear load over the mortar joint of the core samples. The magnitudes of the shear and compression stresses acting on the mortar joint during the test is determined by the angle α of inclination of the diametric mortar joint with respect to the horizontal direction, see Section 3.2. Increasing values of α correspond to increasing shear and decreasing normal compression on the mortar during the test. A crucial point before the execution of the experiments was the decision of the inclinations of the diametric mortar joints for the different Brazilian tests. The range of inclinations must be carefully chosen in order to avoid the vertical splitting of the core sample and to trigger the frictional failure in the mortar [23]. Since the mortar had rather low strength, the Brazilian tests were cautiously carried out with lower inclinations than in previous studies [22–24], i.e. $\alpha = 40^{o}$, $\alpha = 45^{o}$ and $\alpha = 50^{o}$. Five core samples were assigned to each one of these inclinations.

After the extraction of the core samples, the remains of the masonry walls were dismantled with the aim of extracting mortar layers. Small mortar samples with dimensions roughly 50 × 50 × 15 mm^3 were cut from the layers to be tested according to the Double Punch Test (DPT), see Figure 4c. This test was carried out following the DIN 18555-9:1999 standard [30] and relevant references [22,24,31,32]. The specimens were placed centrally between the 20 mm diameter loading platens. The irregular surfaces of the joints were regularized by using gypsum powder. The testing machine had a load cell with 10 kN capacity and the loading rate was 0.0094 kN/s.

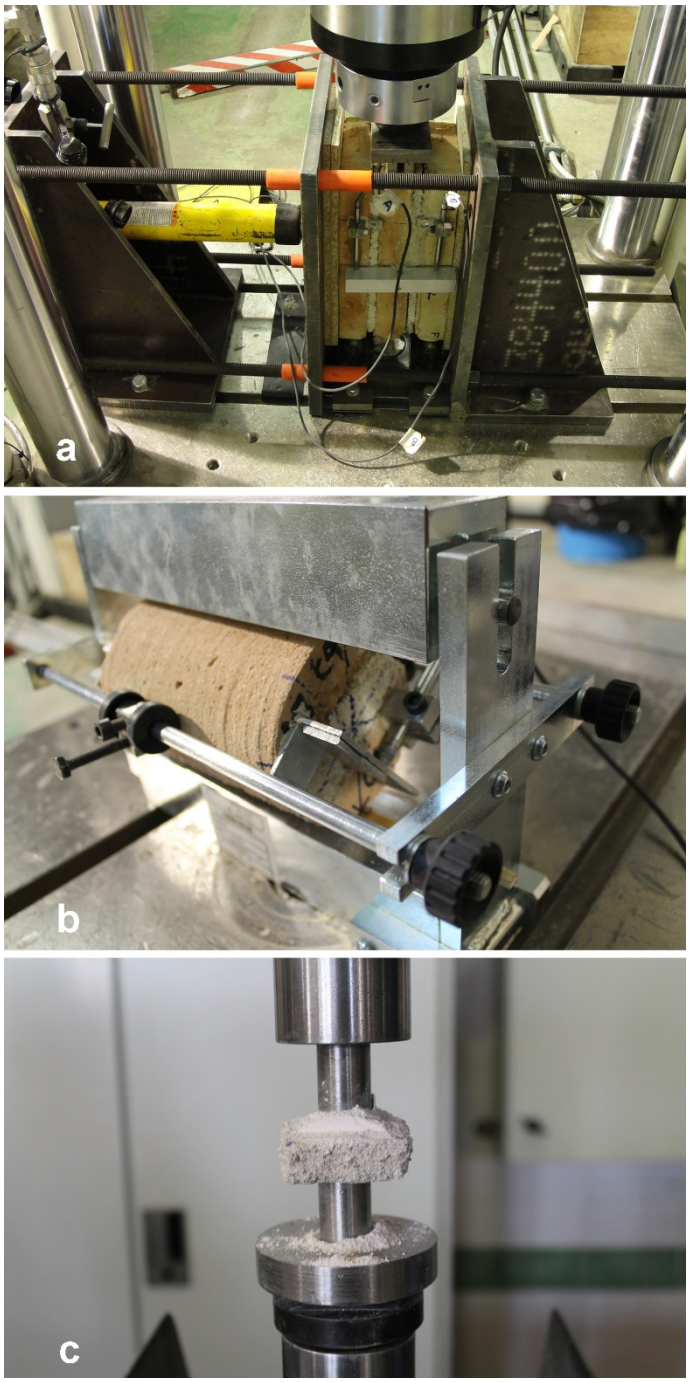

Figure 4 – Experimental setups of the different tests: a) shear tests on triplets, b) Brazilian tests on core samples and c) double punch tests on mortar joints.

3. Experimental results

This section presents the results of the standard shear tests on triplets and the results of the non-standard Brazilian tests on core samples complemented with double punch tests on mortar joints. The outcomes of the experimental program are processed in order to evaluate the shear behaviour of the investigated masonry.

3.1 Shear tests on triplets

According to the EN 1052-3:2002 [12], the normal and shear stresses can be supposed as uniformly distributed over the two bed joints during the triplet test. The maximum value of the experimental shear stress τ_{max} on the bed joints can be thus evaluated as follows:

$$\tau_{max} = \frac{F_{max}}{2A} \quad (1)$$

where F_{max} is the maximum shear force recorded at failure and A is the total area of the bed joint equal to the cross-section of the unit.

Tables 3-4-5 show the experimental results from the shear tests on triplets. Each table reports the images of each specimen before and after the test, the sketch of the failure mode, the values of F_{max} and τ_{max}. The progressive increase of the normal pre-compression load σ_p from 0.3 MPa to 0.6 MPa and 1.0 MPa corresponds to an increase of the maximum shear force due to the frictional behaviour of the material, as expected. Most of the specimens exhibited a sliding failure at the brick-mortar interfaces, although the cracks crossed the mortar material of the joint, through a fracture propagating from one interface to the opposite one. The CV resulted higher in the set of triplets with lowest pre-compression (σ_p = 0.3 MPa), as also observed in previous works [8,33].

The interpretation of the experimental results from triplet tests was carried out according to previous studies [8,10]. The Mohr-Coulomb shear failure envelope can be represented by the straight-line with equation:

$$\tau = C - \sigma \tan \Phi \quad (2)$$

where C is the cohesion and Φ is the internal friction angle. These two parameters can be evaluated by using a linear regression of the experimental results depicted in the σ–τ plane. The experimental results are the points of coordinates (σ_p, τ_{max}). Figure 5 shows the failure envelope derived from the triplet tests, with Mohr-Coulomb parameters resulting Φ = 35.65° and C = 0.04 MPa. The coefficient of determination is R^2 = 0.97, proving the good representativeness of the linear interpolation of the experimental results.

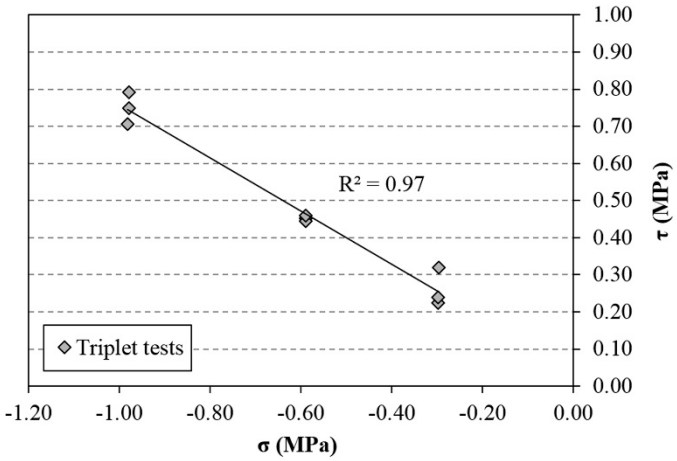

Figure 5 – Interpretation of shear tests on triplets: failure envelope (Φ = 35.65º and C = 0.04 MPa).

3.2 Brazilian tests on core samples and double punch tests on mortar joints

The compressive strength derived from the double punch test can be calculated by the following expression:

$$f_{cm,DPT} = \frac{F_{max}}{\pi r^2} \tag{3}$$

where F_{max} is the maximum load reached during the experiment and r = 10 mm is the radius of the tested surface.

The double punch tests on the joints extracted from the walls provided an average strength value $f_{cm,DPT}$ = 0.91 MPa with CV = 24%, see Table 6. The scattering of results is rather typical for this kind of test due the inevitable heterogeneity of samples that is related to variations of the geometry and to the condition of the mortar bed joint inside the wall. The samples presented variable thicknesses between 17 mm and 21 mm, produced by the irregularities of the units. The double punch tests provided a mean strength lower than that of the uniaxial compression test of prisms (f_{cm} = 1.63 MPa) presented in Section

2.1 and Table 1. This might seem a contradiction since the standard tests of prisms on 40 mm thick samples, in principle, should provide an average compressive strength lower than the double punch tests on thinner joints. This discrepancy can be explained by the different curing conditions and water contents of the two types of mortar sample. Aerial lime hardens when exposed to air and therefore the exposed surface of the sample is an extremely important parameter. The mortar joints presented a much smaller exposition than the 160 × 40 × 40 mm^3 prisms and thus their evolution of strength was slower due to their lower contact with the air. Also, the materials in contact with mortar were different in the two cases, i.e. steel walls of the moulds for prismatic specimens and brick surfaces for the joint samples. The initial moisture content of bricks, at the stage of the construction of the walls, may have also affected the time evolution of the mechanical properties of mortar in the joints. Due to the different curing conditions, mortar prisms cannot represent satisfactorily the physical and mechanical properties actually attained by the mortar bed joints embedded in masonry.

Tables 7-8-9 show the experimental results from the Brazilian tests on core samples with different inclinations of the diametric mortar joint, i.e. $\alpha = 40^o$, $\alpha = 45^o$ and $\alpha = 50^o$. Each table reports the images of each specimen before and after the test, the sketch of the failure mode, the values of the maximum force at failure F_{max} and their corresponding normal stress σ_{max} and tangential stress τ_{max} to the mortar joint. They can be evaluated as follows [22–24]:

$$\sigma_{max} = \frac{F_{max}}{d \cdot l} \cos\alpha \qquad \tau_{max} = \frac{F_{max}}{d \cdot l} \sin\alpha \tag{4a,b}$$

where $d \cdot l$ provides the total area of the diametric mortar joint, d is the diameter of the core sample (90 mm), l is its depth and α is the angle of the inclination of the diametric mortar joint with respect to the horizontal direction. The inclination angle α determines the magnitude of the normal and tangential stresses applied on the mortar joint. Its increase produces an increase of the shear component and a decrease of the normal component of the load applied during the Brazilian test.

The experimental results for $\alpha = 40^o$ were more scattered (CV = 34%) than those for other inclinations, with average F_{max} = 7.89 kN. Two core samples (C2 and C15, see Table 7) exhibited a failure with sliding along the brick-mortar interface, corresponding to lower ultimate loads than those of specimens yielding a failure involving mainly the mortar joint (C20, Ci and especially Cf that provided a far higher ultimate load). This second type of failure was termed “parasymmetric” or “central symmetric” in a previous work by the authors [23] since the fracture crosses the mortar in the centre and propagates towards opposite extremities along the upper/lower brick-mortar interfaces. The last case was observed in previous studies on the Brazilian test on cores with inclined mortar joint.

The experimental average failure load for core samples with $\alpha = 45^{o}$ was F_{max} = 7.56 kN with small scattering of results (CV = 10%). Three samples showed the sliding failure along the brick-mortar interface (C4, C18 and CA), whereas two samples (C9 and C17) exhibited the parasymmetric failure with higher resistance, see Table 8. Compared to the previous case $\alpha = 40^{o}$, it is worth noticing that the shear component of the stress on the mortar joint is increased and this explains the reduction of the average value of the failure load.

The last group of Brazilian tests with $\alpha = 50^{o}$ provided much lower failure loads than in the other inclinations. The average value was F_{max} = 4.44 kN with CV = 20%. The observed failure modes were interface shear sliding in one sample (C12) and in all the others combined interface sliding, fracture involving mortar and splitting of a brick wedge, see Table 9. It has to be noticed that the failure load obtained for the core sample C12 is rather low compared to the other results, in agreement with previous observations that demonstrate that the interface shear sliding is the weakest failure mechanism for core samples subjected to the Brazilian tests.

The results of the Brazilian tests with varying inclination of the mortar joint interposed between two brick cylindrical segments can be interpreted by using the micromechanical approach proposed in [23] that is based on two different procedures. The first one is called *interface model* and it analyses the strength behaviour of the diametric mortar joint as that of a plane interface. From the theoretical point of view, this model seems appropriate to describe the ultimate behaviour of mortar joints exhibiting a failure mechanism with pure sliding along the brick–mortar interface. The ultimate conditions of core samples with different inclinations of the diametric mortar joints can be represented in the σ–τ plane by points with coordinates (σ_{max}, τ_{max}) calculated according to Equations 4a-b. The linear regression of the experimental results provides the Mohr-Coulomb parameters of Equation 2, i.e. the same adopted for the triplet tests. Figure 6 shows the failure envelope derived from the Brazilian tests executed in this research, yielding $\Phi = 35.86°$ and $C = 0.09$ MPa. The coefficient of determination was $R^2 = 0.92$ proving an acceptable representativeness of the linear interpolation of the experimental results, given the inevitable scattering of the Brazilian tests.

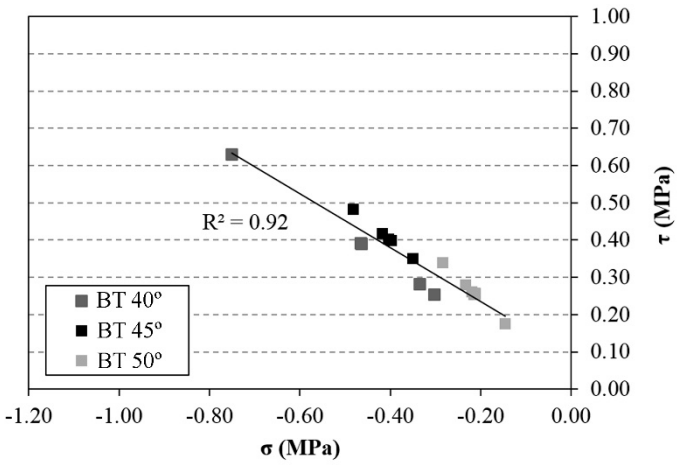

Figure 6 – Interpretation of Brazilian tests on core samples: failure envelope obtained by the interface model (Φ = 35.86º and C = 0.09 MPa).

The second interpretative theory proposed in [23] is the *Continuum Model* that assumes the ultimate strength condition of the specimen due to the shear failure of the material composing the joint, in this case aerial lime mortar. The stress state at failure for each Brazilian test is represented by Mohr's circles in the σ–τ Mohr's plane. The Continuum Model considers also the definition of the Mohr's circles representing the failure stress state of double punch tests. This approach allows the combination of different test methods within the same interpretative micromechanical theory [24]. The failure condition would be ideally given by a Mohr-Coulomb linear criterion becoming tangent to all the Mohr's circles representing the specimens' ultimate stress states in the σ–τ Mohr's plane. Due to the experimental scattering, the empirical Mohr-Coulomb failure envelope is obtained analytically by a least squares method to minimise the sum of the squares of the distances of each Mohr's circle from the linear failure domain.

Figures 7a-b show the construction of the Mohr's circles corresponding to the Brazilian tests and double punch tests. In the first case, the stress state in the mortar is shear-compression with stress components evaluated according to Equations (4a,b). In the second case, the stress state is not considered uniaxial due to the small thickness of the joint sample and the confinement exerted by the punches, as well as by the mortar material surrounding the loaded area. A previous numerical study by the authors evaluated the amount of the transversal confinement as the 5% of the vertical stress [24]. The Continuum Model is thus able to provide a joint interpretation of Brazilian tests on core samples and double punch tests on joint samples within the same theoretical framework, in order to evaluate the strength of the investigated mortar. The Mohr-Coulomb failure envelope of mortar subjected to different shear-compression combinations is expressed as follows:

$$|\tau| = c - \sigma \tan\phi \qquad (4)$$

where c is the cohesion of mortar and ϕ is its internal friction angle. According to the Mohr-Coulomb's theory, the material reaches the failure when the Mohr's circle representing its stress state becomes tangent to the lines given by Equation 4. Given a set of several experimental results, together with their corresponding Mohr's circles built as shown in Figures 7a-b, the values c and ϕ of the overall failure envelope are those that minimize the sum of the squares of the distances between the failure envelope and the Mohr's circles. Figure 7c shows the failure envelope obtained thanks to the Continuum Model applied to the experimental results from double punch tests and Brazilian tests. The Mohr-Columb parameters result $\phi = 26.35^{o}$ and $c = 0.32$ MPa. According to Mohr-Coulomb's theory, it is possible to evaluate the corresponding values of the compressive and tensile strength of mortar f_{cm} and f_{tm} using the two following expressions:

$$f_{cm} = \frac{2c\cos\phi}{1-\sin\phi} \qquad f_{tm} = \frac{2c\cos\phi}{1+\sin\phi} \qquad (5a,b)$$

These equations provide $f_{cm} = 1.02$ MPa and $f_{tm} = 0.39$ MPa. The first value provides a good estimation of the average compressive strength of mortar obtained from the double punch tests ($f_{cm,DPT} = 0.91$ MPa, see Table 6). The tensile strength derived from Mohr-Columb theory results correctly lower than the flexural strength obtained from tests on prismatic specimens ($f_{fm} = 0.55$ MPa, see Table 1).

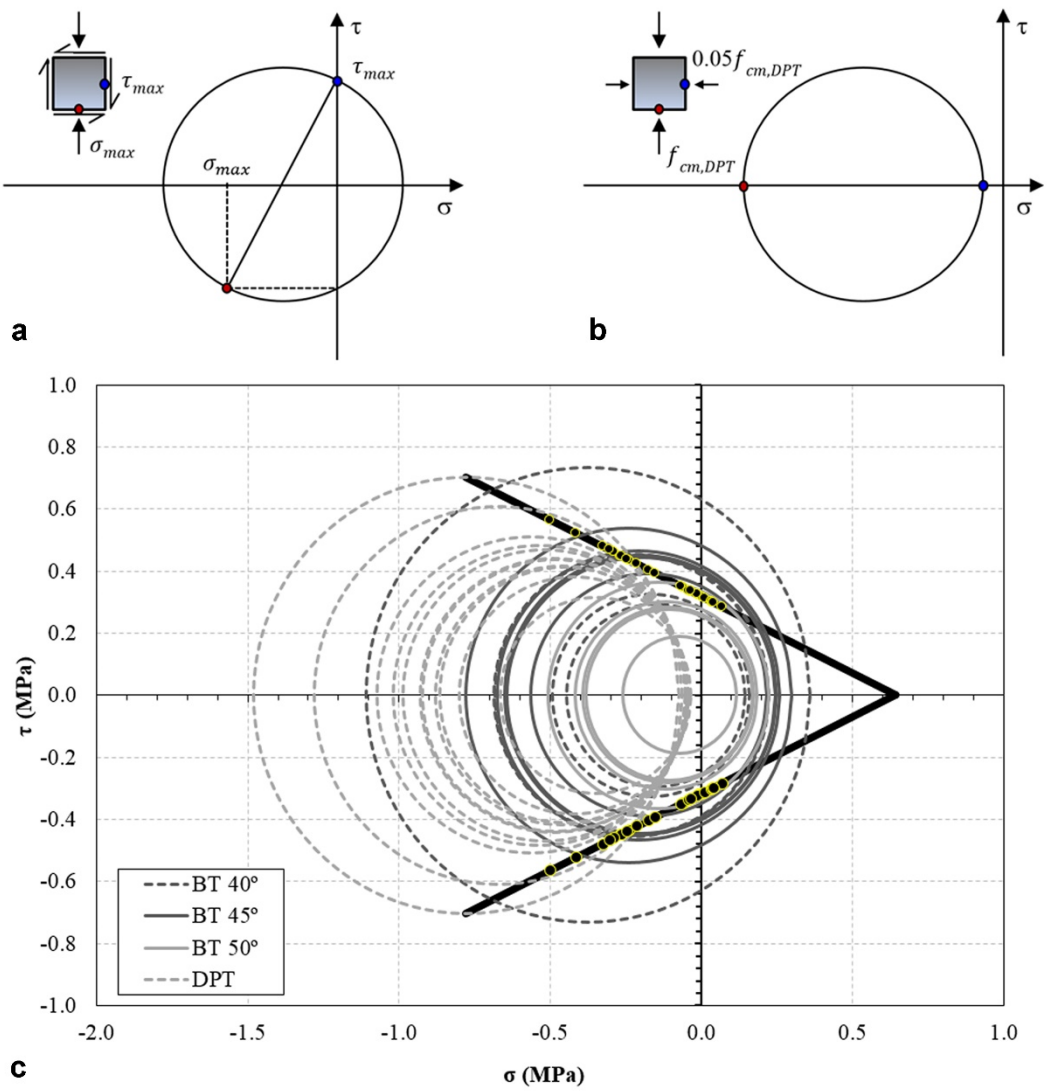

Figure 7 – Interpretation of Brazilian tests on core samples and double punch tests on mortar joints: construction of Mohr's circle representing the stress state of mortar at failure a) in Brazilian tests and b) in double punch tests; c) failure envelope obtained by the continuum model (ϕ = 26.35º and c = 0.32 MPa).

4. Discussion

Both the standard shear tests on triplets and the non-standard Brazilian tests on core samples with inclined diametric mortar joint (complemented with DPT results) provided the shear failure envelopes of the structure composed of the mortar joint and the bond interface. These envelopes were derived from the application of either the interface model or the continuum model. From a theoretical point of view, the interface model seems more compatible with the bond failure at the unit-mortar interface, whereas the continuum model seems adequate to describe the shear failure in the mortar material filling the joint.

In the shear tests on triplets the failure mechanism was the sliding at the bond interface, proving the appropriateness of processing the experimental results through the interface model. The Mohr-Coulomb parameters resulted $\Phi = 35.65°$ and $C = 0.04$ MPa.

On the other hand, the non-standard Brazilian tests exhibited two different types of failures, i.e. either the sliding along the bond interface or the parasymmetric mechanism with shear failure of the mortar material inside the joint. For the interpretation of the non-standard tests that resulted heterogeneous as for the experimental failure mechanisms, the authors have chosen to apply both the interface and continuum models to the whole sample of experimental results. Then, the results from the two different approaches have been critically compared. From a theoretical point of view, it might seem more rigorous to assign the results with sliding failure to the interface model and those with parasymmetric failure to the continuum model. However, this approach could result cumbersome since it would reduce the experimental redundancy of tests assigned to a specific interpretative theory. Redundancy of experimental results is indeed extremely important in experimental campaigns on historical masonry, where the number of available samples is usually limited and the scatterings of the experimental results may result even higher than those observed in this experimental program performed under controlled laboratory conditions. For all these reasons, a better option seems the consideration of both the interface and the continuum models to be applied separately to the whole set of experimental results and then critically compared. Actually, and due to the fact that some of the failures obtained involve both mortar cracking and interface sliding, the real physical phenomenon is a mixed condition between those represented by the two aforementioned insights.

In this experimental program, the interface model applied to the Brazilian tests provided $\Phi = 35.86°$ and $C = 0.09$ MPa, i.e. Mohr-Coulomb parameters quite close to those obtained from triplet tests. The dimensions of the core samples are smaller than those of triplets and this fact produces inevitably a higher experimental scattering since the Brazilian test results depend more on the local conditions of the material components in the small specimen. The coefficients of determination results $R^2 = 0.97$ for standard shear tests triplets and $R^2 = 0.92$ for Brazilian tests on core samples. The discrepancy between the test on triplets and the Brazilian tests was expected due to the different testing setups and natures of the samples and in any case it shows that the interface model can be acceptably applied to both the standard and non-standard shear tests.

On the other hand, the continuum model applied to the Brazilian tests and the DPTs provided $\phi = 26.35°$ and $c = 0.32$ MPa, that are parameters obviously completely different than those of the interface model since they were derived from a different failure theory based on continuum mechanics.

However, when these parameters are inserted into Equations 5a-b they provide compressive and tensile strengths in remarkable agreement with those obtained from direct tests on mortar.

On the basis of these observations, it is highly advisable in practical applications to compare different testing techniques and interpretative models so as to characterize the mechanical properties of existing materials.

5. Conclusions

This research has focused on the evaluation of the shear strength of aerial lime mortar brickwork. This kind of material has so far deserved very little attention in the existing literature. The limited research on the mechanical properties of aerial lime mortar is due to the practical difficulties caused by its long curing and hardening time. The experimental program presented in this research has considered masonry samples built in the laboratory and then tested one year after their construction.

The research has analysed two different testing techniques to evaluate the shear strength of aerial lime mortar brickwork. The first one is the triplet test established by the EN 1052-3:2002 [12]. Being a standard procedure, this technique presents a straightforward interpretation of the experimental results and can be easily carried out for the mechanical characterisation of new masonry. However, this approach cannot be applied to the investigation of the shear strength of existing masonry structures, since it is practically impossible to extract this type of sample from the existing structural member. For this reason, the present study has investigated a second testing technique based on the in-situ core drilling and subsequent testing of masonry cylindrical specimens in the laboratory. Masonry walls were built to reproduce historical masonry members and then they were core drilled to extract cylindrical specimens with 90 mm diameter. These core samples were subjected to different Brazilian tests with varying inclinations of the diametric mortar joint. This non-standard MDT procedure can be easily applied to existing structures by combining several advantages like low invasivity, easy sampling and limited costs, as demonstrated by recent works [22–24].

Both standard and non-standard testing methods have been investigated on the same materials and under controlled laboratory conditions in order to assess and validate their applicability. The following conclusions can be drawn at the end of the experimental work:

- The comparative study of standard and non-standard shear tests has allowed the evaluation of the strength of existing masonry through a novel MDT technique based on the extraction of small size core samples. This approach can be considered a suitable alternative to more

destructive approaches, such as the in situ diagonal compression test, aimed at the mechanical characterisation of the shear strength of existing masonry.

- The Brazilian tests on core samples have shown their utility for the estimation of the material shear strength. Two different interpretation theories have been presented, called interface and continuum models. They correspond respectively to a purely frictional sliding model over the mortar-unit bond interface and a Mohr-Coulomb failure criterion for the mortar. The first interpretation considers only the results from the Brazilian tests, whereas the second approach considers both the Brazilian tests and DPT results of mortar joints extracted from the walls. This second option shows to be appropriate when it is required a combined interpretation of experimental results from different testing methods, as usual in the analysis of existing structures.
- Due to the fact that some of the failures involve mainly mortar cracking and others mainly interface sliding, the ultimate mechanism in Brazilian tests is not unique and both the interpretative theories have been applied and critically compared. The interface model has provided a shear failure envelope in good agreement with that derived from standard tests on triplets. The parameters obtained from the continuum model based on Mohr-Coulomb theory, on the other hand, have provided strength values in very good agreement with those previously obtained from direct tests on mortar samples.
- The comparison between the standard and non-standard testing techniques has revealed to be of high importance in order to deepen the understanding of the MDT procedures for the characterisation of the shear strength of existing brickwork. The investigation of masonry composed of solid clay bricks and aerial lime mortar joint has provided insight on the performance of the materials typically found in historical heritage structures.

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Tables

Table 1 – Experimental strengths of aerial lime mortar prisms

Mortar CL90 Sample	f_{fm} (MPa)	f_{cm} (MPa)
Prism 1	0.48	1.70 1.54
Prism 2	0.52	1.47 1.72
Prism 3	0.60	1.63 1.72
Prism 4	0.60	1.68 1.58
Average	0.55	1.63
CV %	9.5	5.3

Table 2 – Experimental strengths of bricks

Brick Sample	$f_{cb,u}$ (MPa)	$f_{cb,c}$ (MPa)	$f_{fb,x}$ (MPa)	$f_{fb,y}$ (MPa)
1	31.2	19.6	3.81	3.81
2	29.4	19.5	3.67	3.53
3	30.4	19.1	3.60	3.57
4	30.3	18.3	3.52	3.74
5	34.2	16.6	3.51	3.19
6	28.9	17.5	3.59	3.32
7			3.88	4.40
8			3.48	4.73
Average	30.7	18.4	3.63	3.79
CV %	5.6	5.9	3.7	13.1

Table 3 – Experimental results of shear tests on triplets with σ_p=0.3 MPa

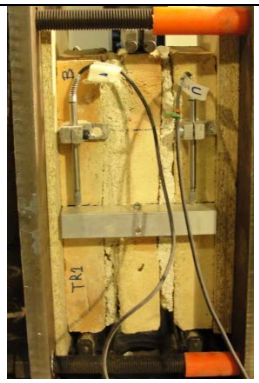
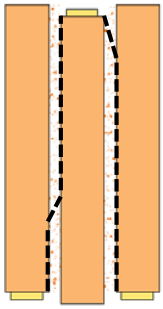
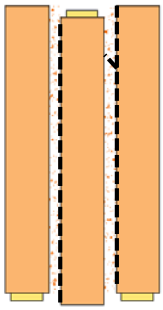
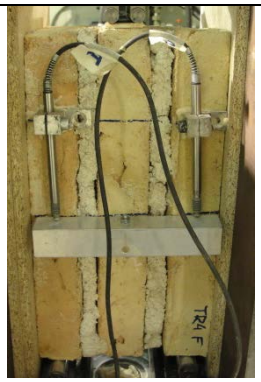
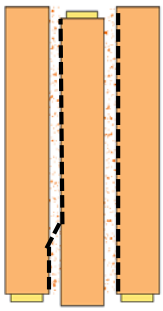
ID	Before Test	After Test	Failure Mode	F_{max} (kN)	τ_{max} (MPa)
TR1				27.73	0.32
TR2				19.50	0.22
TR4				20.75	0.24
Average				**22.66**	**0.26**
CV				**16%**	**16%**

Table 4 – Experimental results of shear tests on triplets with σ_p=0.6 MPa

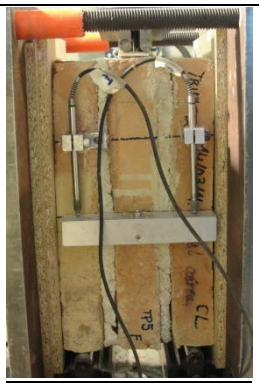
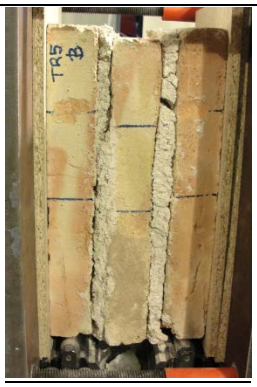
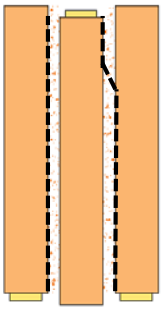
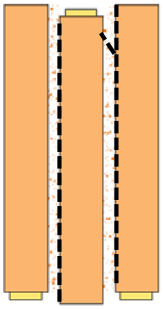
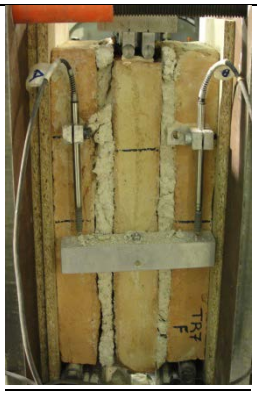
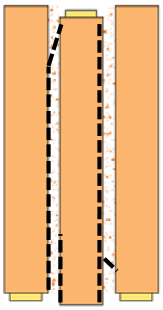
ID	Before Test	After Test	Failure Mode	F_{max} (kN)	τ_{max} (MPa)
TR5				39.26	0.45
TR6				38.64	0.44
TR7				39.99	0.46
Average				**39.30**	**0.45**
CV				**1%**	**1%**

Table 5 – Experimental results of shear tests on triplets with σ_p=1.0 MPa

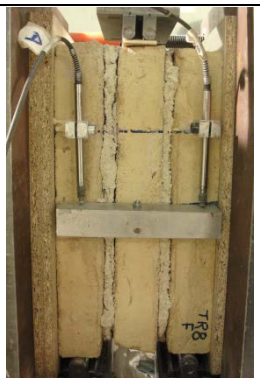
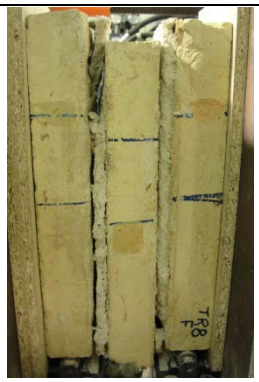
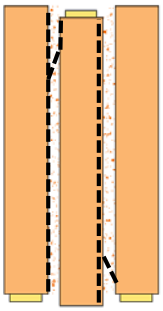
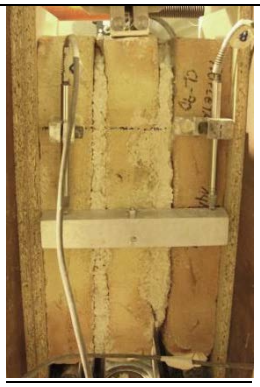
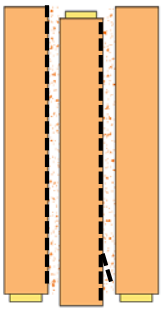
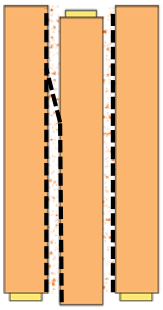
ID	Before Test	After Test	Failure Mode	F_{max} (kN)	τ_{max} (MPa)
TR8				65.12	0.68
TR9				68.9	0.72
TR10				61.33	0.64
Average				**65.12**	**0.68**
CV				**5%**	**5%**

Table 6 – Experimental results of double punch tests on extracted mortar joints

Joint Sample	$f_{cm,DPT}$ (MPa)
1E_CL	0.87
2E_CL	1.48
3E_CL	0.80
4E_CL	0.93
5E_CL	1.28
6E_CL	0.92
7E_CL	1.07
8E_CL	0.88
9E_CL	0.99
10E_CL	0.67
11E_CL	1.02
Average	0.91
CV %	24.0

Table 7 – Experimental results of Brazilian tests on core samples with α=40º

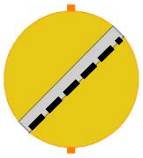
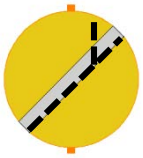
ID	Before Test	After Test	Failure mode	F_{max} (kN)	σ_{max} (MPa)	τ_{max} (MPa)
C2				5.699	0.335	0.281
C15				5.138	0.302	0.253
C20				7.925	0.465	0.390
Ci				7.889	0.463	0.389
Cf				12.785	0.750	0.629
Average				**7.887**	**0.463**	**0.388**
CV (%)				34.0	34.0	34.0

Table 8 – Experimental results of Brazilian tests on core samples with α=45º

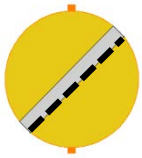
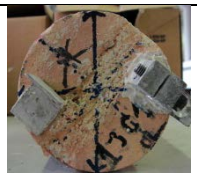
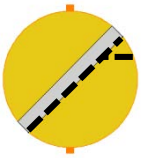
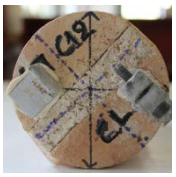
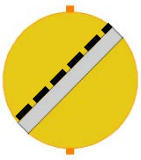
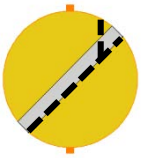
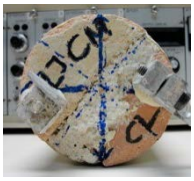
ID	Before Test	After Test	Failure Mode	F_{max} (kN)	σ_{max} (MPa)	τ_{max} (MPa)
C4				7.334	0.397	0.397
C9				8.893	0.482	0.482
C17				7.696	0.417	0.417
C18				7.424	0.402	0.402
CA				6.449	0.349	0.349
Average				**7.559**	**0.410**	**0.410**
CV (%)				10.0	10.0	10.0

Table 9 – Experimental results of Brazilian tests on core samples with α=50º

ID	Before Test	After Test	Failure Mode	F_{max} (kN)	σ_{max} (MPa)	τ_{max} (MPa)
C3				5.746	0.283	0.337
C12				2.957	0.146	0.278
Cb				4.732	0.233	0.173
Ch				4.436	0.213	0.254
Cm				4.324	0.219	0.261
Average				**4.439**	**0.219**	**0.261**
CV (%)				20.0	20.0	20.0