Cylindrical samples of brick masonry with aerial lime mortar under compression: experimental and numerical study

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Abstract – This research presents an experimental and numerical study focusing on the compression test of cylindrical samples core drilled from existing masonry walls. This method is suitable for the minor destructive assessment of the mechanical properties of historical masonry, like that composed of aerial lime mortar joints and solid clay bricks. This particular material combination, frequently found in the vast majority of the built cultural heritage, was utilized to build representative specimens that were stored in the laboratory for one year until their testing. Cylindrical samples of 150 mm diameter were extracted from the masonry walls by using a dry core-drilling procedure, and then regularized to be tested under compression in the laboratory. A comparison is presented between the experimental results on cylindrical samples and those obtained from standard compression tests on prismatic samples consisting in stack bond prisms. Numerical simulations by finite element micro-modelling of the compression tests on the core samples were carried out to investigate the experimental behaviour of the specimens and evaluate the compressive strength of the material from this nonstandard technique. The combined experimental and numerical study allows the assessment of important mechanical parameters for the compressive characterisation of masonry composed of aerial lime mortar joints and solid clay bricks.

Keywords: Brickwork; Coring; In-situ; Sampling; Minor Destructive Testing (MDT); Compressive Strength; Finite Element Method; Micro-modelling; Continuum Damage Mechanics; Damage.

Abbreviations - MDT: Minor Destructive Testing; 2JC: Two-Joint Cylinder; 3JC: Three-Joint Cylinder; LVDT: Linear Variable Differential Transformer, DPT: Double Punch Test.

1. Introduction

The knowledge of the compressive behaviour of masonry is essential in the current design practice [1]. However, the assessment of the compressive mechanical properties of masonry can be rather challenging in studies involving the assessment of existing
historical buildings due to the need to minimize possible damage induced to the structure by the inspection procedures. 

For this type of constructions, a rather recurrent material is the combination of lime mortar joints and solid clay bricks. Experimental studies about this type of masonry are of paramount importance for people involved in the analysis and restoration of historical structures. However, the possibility of laboratory tests on newly manufactured lime mortar samples encounters a significant difficulty in the long curing and hardening periods [2–4]. In spite of this difficulty, some methods have been proposed to characterise masonry as a composite material based on the mechanical properties of the material components, i.e. lime mortar and brick units. The analysed masonry samples are usually prismatic as those required by current technical standards [5,6], e.g. stack bond prisms [7,8], small columns [9,10] and Flemish bond wallets [8,11–13].

More recent works focus on laboratory testing of cylindrical samples core-drilled from lime mortar and clay brick masonry walls [14–17]. This minor destructive testing (MDT) method was suggested by the UIC 778-3 recommendations [18] and has shown to be suitable for the assessment of the mechanical properties of existing structures due to its limited invasiveness [19–21]. The extraction of cylindrical samples from existing masonry walls is more appropriate than cutting prismatic wallets, as those recommended by standards for new construction [5,6]. The main reason is that core drilling provides undamaged cylindrical masonry samples, easy to transport and maintain intact until the delivery to the laboratory. This favourable feature can ensure a reliable evaluation of compressive strength from undisturbed specimens. On the contrary, the prismatic wallets cut from existing brickwork are normally prone to damage during the extraction operations, consisting typically in the combined use of a circular saw and a chisel, as well as during the transport to the laboratory.

There is, however, the need for experimental criteria to correlate the results obtained from this novel non-standard method with those referring to the standard compression tests on prismatic samples. For this reason, reference laboratory tests on walls replicating historical-like materials seem necessary to gain further knowledge about the correct execution of core sampling, experimental setup and testing, results processing and derivation of mechanical parameters for the compressive characterisation. In particular, benchmark studies are needed about the application of this MDT technique to aerial lime mortar and solid brick masonry, a traditional material of the built heritage. The available numerical methods, such as those based on finite element micro-modelling [22,23], may
also help to interpret the mechanical behaviour of masonry cylindrical specimens by advanced computational simulations of their resisting mechanism under compressive loading.

The main objective of this research is to provide comprehensive experimental results supported by numerical simulations about the compressive behaviour of cylindrical specimens of aerial lime mortar and terracotta brick masonry. Compression tests were carried out by considering cylindrical samples core-drilled from walls replicated in the laboratory, together with standard stack bond prisms. The extraction of cylindrical specimens was executed one year after the construction of the walls, by using a dry core drilling technique to avoid disjointing between low strength mortar joints and units. The regularization of cylindrical samples was carried out by means of high-strength mortar caps [14,15] in order to allow the application of the compressive load over the investigated material. Two different cylindrical specimens were tested, including either four brick pieces (with two horizontal mortar joints and a vertical one) or three brick pieces (with two horizontal mortar joints). The results from compression tests on core samples and prismatic specimens are finally compared in an attempt to provide experimental correlations among the different types of specimens.

The experimental calibration of this MDT technique is pursued in order to provide reliable procedures for its possible application to existing structures of the built heritage. The presentation of advanced numerical simulations by finite element micro-modelling aims to give insights into the experimental behaviour of the extracted cylindrical specimens when tested under compression, regarding their resisting mechanism and type of failure.

2. Experimental program

The experimental program was executed at the Technical University of Catalonia (UPC-BarcelonaTech) in the Laboratory of Technology of Structures and Building Materials. This section presents in detail the characterisation of the material components, the masonry specimens and the testing setups.

2.1 Characterisation of construction materials

One of the most important aspects of the experimental campaign was the choice of the material components for building the specimens. In order to reproduce historical materials, pure aerial lime mortar CL90 without cement [24] and handmade terracotta
bricks were used (Figure 1). The mortar was prepared by using a volume ratio of binder to aggregate of 1:3 (lime:sand), while the content of water was defined by the workmanship in order to achieve an optimum workability of the mixture. The adopted silica sand had a grain size distribution of 0 ÷ 2 mm.

The mortar’s flexural ($f_{fm}$) and compressive ($f_{cm}$) strengths were measured on prisms with dimensions $160 \times 40 \times 40 \text{ mm}^3$ (Figure 1a) according to [25]. Four specimens were provided for the flexure test. Eight halves, resulting from the prisms splitted after the four flexure tests, were tested under compression with $40 \times 40 \text{ mm}^2$ steel loading platens (Figure 1c). The tests were performed with a 10 kN load cell under load control. The rate of the applied load was constant during each test with values of 10 N/s for the flexure tests and 50 N/s for the compression tests. Table 1 presents the results obtained one year after the pouring of mortar. The average value of flexural strength was $f_{fm} = 0.55 \text{ MPa}$ and that of compressive strength was $f_{cm} = 1.63 \text{ MPa}$, with coefficients of variation (CV) of 9.5% and 5.3% respectively. These results seem reasonable if compared with the strength values from previous studies on lime mortar available in the literature [2,3].

Figure 1 – Flexure and compression testing of prismatic samples of mortar (a,c) and brick (b,d). Double punch testing of mortar joints (e).
Table 1 – Experimental flexural and compressive strengths derived from prisms of aerial lime mortar.

<table>
<thead>
<tr>
<th>Mortar CL90 sample</th>
<th>$f_{cm}$ (MPa)</th>
<th>$f_{cm}$ (MPa)</th>
</tr>
</thead>
<tbody>
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<td>1</td>
<td>0.48</td>
<td>1.70</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.54</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>1.72</td>
</tr>
<tr>
<td>3</td>
<td>0.60</td>
<td>1.63</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.72</td>
</tr>
<tr>
<td>4</td>
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<td></td>
<td></td>
<td>1.58</td>
</tr>
<tr>
<td>Average</td>
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<td>1.63</td>
</tr>
<tr>
<td>CV %</td>
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<td>5.3</td>
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</tbody>
</table>

Masonry specimens were built using handmade terracotta bricks with nominal dimensions $305 \times 145 \times 45$ mm$^3$. Six units were tested under compression according to [26], with a machine equipped with a 3000 kN load cell. The average value of the compressive strength of units was $f_{cb,u} = 30.7$ MPa with CV=5.6%. The normalized uniaxial compressive strength of the bricks resulted 21.49 MPa, after applying the shape factor equal to 0.7 as recommended by the standard [26]. Six cubes cut from the units were also tested in compression (Figure 1d), providing an average value of the compressive strength of $f_{cb,c} = 18.4$ MPa, with CV=5.9%. The difference between the average compressive strengths measured in the entire units and in the smaller cubes is due to the very different aspect ratios of the two specimens [27], as the entire brick is subjected to much higher transversal confinement exerted by the loading plates of the testing machine than the cube sample. Sixteen brick prisms were tested under flexure to evaluate the flexural strengths along the header and stretcher directions (Figure 1b), providing average values $f_{fb,x} = 3.63$ MPa (CV=3.7%) and $f_{fb,y} = 3.79$ MPa (CV=13.1%), respectively. The reference standard considered was the same adopted for mortar prisms [25] due to the lack of specific guidelines for flexure test of brick samples. The experimental results for brick samples are summarised in Table 2. These results seem reasonable if compared with the strength values from previous studies on solid clay bricks available in the literature [11–13].
Table 2 – Experimental compressive and flexural strengths of bricks

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

The mechanical parameters derived from standard tests show that the present experimental campaign adopted component materials with low-to-moderate properties, especially for the lime mortar [2,3]. The handmade terracotta units, due to their traditional manufacturing process, provided strength values much lower than the modern bricks produced for the construction of new masonry [23].

2.2 Construction of masonry specimens

Two walls with dimensions $1605 \times 870 \times 145$ mm$^3$ were built in stretcher bond (Figure 2a) using the materials detailed in Section 2.1. The construction was intended to reproduce a wall with low strength mortar, typical of historical masonry buildings. The two walls were constructed in the present research to allow the simulation of in-situ core-drilling of 150 mm diameter cylindrical specimens. The walls were built over a steel beam. At the end of their construction, a regularization layer of mortar was poured over the wall, to permit the location of another steel beam. The top and the bottom steel profiles were then connected by two low-tensioned bars in both faces of the walls. The sole purpose of this pre-compression was to provide sufficient confinement to the walls in order to avoid their damage during the laboratory operations, like transportation of samples and coring.

Six stack bond prisms were built with the same materials during the construction of the walls. The stack bond prisms consisted of five bricks and four mortar bed joints with dimensions $305 \times 305 \times 145$ mm$^3$ (Figure 2b). The top and the bottom surfaces of the prisms were coated with a thin regularization layer of high-strength cement mortar, to ensure the uniform distribution of the compressive load as established by [5].
Figure 2 – Construction of specimens in the laboratory: (a) masonry walls and (b) stack bond prisms.

The operations of extraction of cylindrical samples started after one year of curing, after taking out the walls of their storage. At that moment, the aerial lime mortar showed to have reached a proper level of carbonation, as proved by the use of phenolphthalein indicator solution, as well as a sufficient strength, as derived from the prismatic specimens, see Table 1. The core-drilling was executed using a dry procedure, i.e. without water (Figure 3a). This solution showed several advantages in recent research by the authors [16] compared with the traditional wet procedures that spoil the samples by washing the mortar joints away. The drilling machine induced only a rotation to the diamond bit, avoiding any excess of vibration that may cause the formation of internal cracks within the cylindrical specimen. The position of the extracted cylinders was decided in order to provide as many specimens as possible. Coring operations were carefully executed in order to avoid undesirable disjointing of specimens. The procedure was subdivided into different stages. First, the machine drilled until the half of the thickness of the wall. Second, the dust produced by the drilling was removed from the interior of the core bit and from the prints on the wall. The drilling operation was eventually completed through the whole thickness of the wall. The cylinders were carefully extracted from the core bit and immediately confined with strong adhesive tape stuck on their lateral curved surface. They were finally stored in the laboratory to be tested. Figure 3b shows the remaining wall after the core-drilling. It is worth noticing the
absolute absence of water as well as the reduced amount of dust due to the use of the vacuum cleaner connected to the drilling equipment. Therefore, the operations of extraction of samples proved to be usefully clean.

Three types of specimens were extracted from the walls. First, 150 mm diameter cores with three brick pieces and two horizontal mortar joints, called 2JCs (two-joint cylinders) (Figure 4a). Second, 150 mm diameter cores including four brick pieces, two horizontal mortar joints, and a vertical mortar joint, called 3JCs (three-joint cylinders) (Figure 4b). Finally, 90 mm diameter cores with a mortar diametric joint between two brick cylindrical segments that were employed in another experimental study [17]. The 2JCs and the 3JCs were used for the compression test following the procedures presented in [14,15]. Even
though fifteen 2JCs and fifteen 3JCs were extracted successfully, compression tests were executed on six 2JCs and six 3JCs.

The 2JCs and 3JCs were regularized with mortar caps of 105 mm width made of high strength mortar, shaped by means of special wooden moulds (Figure 4c). The average value of compressive strength for the regularization mortar after 28 days was 65 MPa and thus much higher than the one expected from masonry core samples. The curved lateral surface of the cylinders was regularized to create two plane surfaces parallel to the bed joints to be loaded by vertical compression. The specimens were stored in the laboratory conditions and, once hardened, extracted from the moulds to be tested (Figure 4d). This regularization ensures an optimal transmission of the compressive force from the testing machine to the specimen through a perfectly adherent high-strength cap, therefore avoiding possible stress concentrations [14]. This specific approach is different from the one proposed by UIC guidelines [18] and other relevant works [20] which used metal concave pieces gripped to the testing machine.

Figure 4 – Cylindrical specimens obtained from core-drilling of the wall: (a) two-joint cylinders, (b) three-joint cylinders, (c) regularization by mortar capping, (d) samples ready for testing.

2.3 Characterisation of mortar joints

Additionally to the compression tests on mortar prisms detailed in Section 2.2, the compressive strength of the aerial lime mortar in the joints of the walls was evaluated
through the double punch test (DPT) following the DIN 18555-9:1999 standards [28] as well as references [18,29]. After the execution of core drilling, the remaining walls with holes in them (Figure 3b) were dismantled very cautiously course by course by using a very small chisel, in order to extract carefully the mortar slabs. Mortar samples with dimensions roughly 50 × 50 × 18 mm³ were extracted from the joints of the masonry walls after the extraction of the core samples. Gypsum powder was used to regularize the irregular faces of the mortar samples, placed between loading platens with 20 mm diameter (Figure 1e). The loading rate of the testing machine was 0.0094 kN/s and the load cell had a capacity of 10 kN.

Table 3 presents the results of the 11 tested mortar joint samples. The average compressive strength measured with the DPT was $f_{cm,DPT} = 0.91$ MPa, with CV=24%. This value is lower than the average compressive strength derived from the standard prismatic specimens (1.63 MPa, see Table 1), even though the joint samples had lower thickness. This difference is due to the diverse curing conditions of the two typologies of mortar specimens. The aerial lime hardens when exposed to air and the mortar joints of the wall had a much lower external exposed surface, with a limited thickness of around 18 mm, than the 160 × 40 × 40 mm³ prismatic mortar samples conforming with the standards [25]. For this reason, the development of the strength in the mortar joints was slower than in the standard prisms. In addition, the joints were built in contact with the bricks’ surfaces, whereas the prismatic samples were casted into steel moulds. The initial moisture of the bricks during the construction of the walls might also influence the evolution of the strength of the joints. All the mentioned factors may also explain the higher CV in DPT tests than in the standard compressive ones, together with the unavoidable geometric irregularity of the joint samples extracted from the walls.

The previous observations suggest that the DPT provided a more realistic evidence of the actual compressive strength reached by the aerial lime mortar joints within the wall, since the compressive strength derived from standard prismatic samples was affected by different curing conditions and thus exhibited different physical and mechanical properties.
Table 3 - Experimental strengths from double punch tests on mortar joints extracted from the walls.

<table>
<thead>
<tr>
<th>Mortar CL90 joint sample</th>
<th>$f_{cm,DPT}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.87</td>
</tr>
<tr>
<td>2</td>
<td>1.48</td>
</tr>
<tr>
<td>3</td>
<td>0.80</td>
</tr>
<tr>
<td>4</td>
<td>0.93</td>
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<tr>
<td>5</td>
<td>1.28</td>
</tr>
<tr>
<td>6</td>
<td>0.92</td>
</tr>
<tr>
<td>7</td>
<td>1.07</td>
</tr>
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<td>8</td>
<td>0.88</td>
</tr>
<tr>
<td>9</td>
<td>0.99</td>
</tr>
<tr>
<td>10</td>
<td>0.67</td>
</tr>
<tr>
<td>11</td>
<td>1.02</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.91</strong></td>
</tr>
<tr>
<td><strong>CV%</strong></td>
<td><strong>24.0</strong></td>
</tr>
</tbody>
</table>

2.4 Testing setups

All the specimens, i.e. core samples and stack bond prisms, were tested under compression one year after their construction [30]. The compression tests of six 2JCs and six 3JCs were executed according to the procedures reported in [14,18]. The tests consisted in applying a compressive loading on the regularization caps perpendicularly to the bed joints (Figure 5a). The compression machine was equipped with a 200 kN load cell. Both the vertical and horizontal displacements were recorded through four linear variable differential transformers (LVDTs). Two vertical LVDTs (±5 mm range and 5 μm precision) were attached to the regularization caps, two horizontal LVDTs (±1.5 mm range and 1.5 μm precision) were placed horizontally in the front faces of the specimens and two horizontal LVDTs (±5 mm range and 5 μm precision) were fixed on two external metal supports along the diametric direction of the specimen. The test was divided into two stages, corresponding to elastic loading/unloading and then loading beyond the failure. The first stage was performed under load control by carrying out three loading/unloading cycles in order to evaluate the elastic response of masonry. The cycles were performed between the 5% and 20% (3 kN ÷ 20 kN) of the originally expected maximum load. The second loading stage was performed under displacement control, at a rate of 0.004 mm/s. This last stage allowed to measure the compressive strength of the sample and to characterise its softening behaviour. The Young’s modulus of masonry was evaluated from the last loading/unloading cycle, using the measurements from the two vertical LVDTs placed on the specimens.
The stack bond prisms were tested under compression loading in order to compare the results with those from the core samples. The standard [6] was followed for the testing procedures. The samples were regularized with a 10 mm layer of high strength mortar on the upper and lower parts to ensure the uniform loading distribution during the test. The compression machine was equipped with a 3000 kN load cell. Six LVDTs were used to measure vertical and horizontal displacements, according to [5] (Figure 5b). Four vertical LVDTs (±5 mm range and 5 μm precision) were stuck on each lateral face of the prism and two horizontal LVDTs (±1.5 mm range and 1.5 μm precision) were stuck on the two larger lateral faces of the prism. Three initial loading cycles under force control were executed between the 5% and 33% (11 kN ÷ 73 kN) of the expected maximum load. The latter was estimated by using the equation suggested by the Eurocode 6 [31] based on the individual strengths of the masonry components. The second loading stage was performed under displacement control, with a rate of 0.008 mm/s, in order to reach the compressive strength and then to follow the post-peak response of masonry.

![Figure 5 – Experimental setups for compression tests on: (a) core samples and (b) stack bond prisms.](image)

### 3. Experimental results

This section summarizes the main results of the experimental campaign. Standard and non-standard compression tests provided a meaningful set of data that can be analysed to evaluate the mechanical parameters of the investigated materials. The outcomes of the tests from all the specimens were used to evaluate the compressive strength and the elastic parameters of the studied masonry.

#### 3.1 Compression tests on core samples

The compressive stress acting on the cylindrical specimens was evaluated considering two different values. The stress value $\sigma_1$ was calculated by considering the total diametric
section of the cylinder, while the stress value $\sigma_2$ was obtained by using the section of the regularization cap. The first approach is the one suggested by the UIC 778-3 recommendations [18] to calculate the compressive strength of masonry from cylindrical specimens. It considers the entire horizontal cross section of the specimen as the resistant section until reaching the maximum load. The second approach has been investigated in previous research by the authors [14,15] and it is also considered herein, in addition to the first one, to evaluate the compressive strength. This second approach is suggested by the experimental evidence of the crack pattern shown by the specimens close to failure. According to this evidence, observed in all tests, the specimens experience cracking and subsequent detachment of the lateral less confined portions of material. Close to failure, the compressive stresses are concentrated in a sand cone shape limited by the cracks developed. As a result, the effective resisting cross-section of the cylinder may be smaller than the total horizontal one.

Figure 6 shows the stress-strain curves calculated for the first stage of loading/unloading for both 2JC and 3JC, as well as the curves obtained for the last loading stage beyond failure. As mentioned, the loading/unloading cycles were used to evaluate the Young’s modulus of the material. They were evaluated making reference to both $\sigma_1$ and $\sigma_2$ values, leading to the definition of the relevant values $E_1$ and $E_2$. As for the evaluation of the compressive strength values, the value $f_{C1}$ was defined considering the maximum force divided by the diametric cross-section of the cylinder, while the value $f_{C2}$ was obtained considering the maximum force divided by the cross-section of the regularization cap. The relevant expressions are:

$$f_{C1} = \frac{F_{max}}{\phi l} \quad f_{C2} = \frac{F_{max}}{bl}$$  \hspace{1cm} (1a,b)

in which $b$ is the width of the mortar cap, $l$ is the length of the cylinder and $\phi$ its diameter.
Figure 6 – Stress vs. strain curves of core samples: elastic loading/unloading cycles for two-joint (2JC) (a) and three-joint cylinders (3JC) (c); loading beyond failure for two-joint (b) and three-joint cylinders (d).

Table 4 presents a summary of the experimental results from compression tests on core samples. As for the values of $f_{C1}$ and $f_{C2}$, the 2JCs provided average values of 4.25 MPa and 6.17 MPa, whereas the 3JCs provided average values of 3.98 MPa and 5.78 MPa, respectively. As for the values of $E_1$ and $E_2$, the 2JCs provided average values of 1182 MPa and 1716 MPa, whereas the 3JCs provided average values of 1323 MPa and 1921 MPa, respectively.

Table 4 – Experimental results of compression tests on core samples in terms of compressive strengths and Young’s moduli.

<table>
<thead>
<tr>
<th>Sample</th>
<th>$f_{C1}$ (MPa)</th>
<th>$f_{C2}$ (MPa)</th>
<th>$E_1$ (MPa)</th>
<th>$E_2$ (MPa)</th>
<th>Sample</th>
<th>$f_{C1}$ (MPa)</th>
<th>$f_{C2}$ (MPa)</th>
<th>$E_1$ (MPa)</th>
<th>$E_2$ (MPa)</th>
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<td>CV %</td>
<td>11.04</td>
<td>11.05</td>
<td>21.4</td>
<td>21.4</td>
</tr>
</tbody>
</table>
Both the 2JCs and 3JCs presented a typical sandglass failure after the compression test (Figure 7). All the specimens, after reaching the peak of the compressive load, exhibited cracks in correspondence with the edges of the regularization cap, which created two lateral wings falling off from the specimens. At the end of the test, the remaining central part of the specimens was hourglass shaped, with upper and lower sections equal to those of the regularization caps, and central one smaller than the extreme ones.

Figure 7 – Typical hourglass failure exhibited by core samples after the compression tests: (a) two-joint and (b) three-joint cylinders.

3.2 Compression tests on stack bond prisms

Figure 8 shows the stress-strain curves calculated for the first stage of loading/unloading for stack bond prisms, as well as the curves obtained for the following monotonic loading stage until failure. The post peak branch was not measured in specimens SBP2, SBP4 and SBP5, as the vertical LVDTs either detached at the maximum force or were removed to avoid their damaging during the completion of the test in the softening stage.
Table 5 shows the summary of the compression tests on stack bond prisms in terms of compressive strength and elastic parameters. The average compressive strength of stack bond prisms was 5.79 MPa, whereas the Young’s modulus was 2014 MPa. The evaluation of the elastic parameters was possible for all the specimens. All the six stack bond prisms showed a similar failure mode. First, cracks appeared in the central bricks and then propagated towards the upper part of the specimens. All the cracks were vertical and crossed almost all the height of the prism, as expected from a compression test. Extensive cracks appeared also in the lateral fragments. Several substantial cracks appeared on both sides of the prisms (Figure 9).
Table 5 – Experimental results of compression tests on stack bond prisms in terms of compressive strengths and Young’s moduli.

<table>
<thead>
<tr>
<th>Stack Bond Prism</th>
<th>$f_c$ (MPa)</th>
<th>$E$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBP1</td>
<td>7.37</td>
<td>2850</td>
</tr>
<tr>
<td>SBP2</td>
<td>4.98</td>
<td>2055</td>
</tr>
<tr>
<td>SBP3</td>
<td>6.05</td>
<td>1098</td>
</tr>
<tr>
<td>SBP4</td>
<td>5.97</td>
<td>1940</td>
</tr>
<tr>
<td>SBP5</td>
<td>6.15</td>
<td>2685</td>
</tr>
<tr>
<td>SBP6</td>
<td>4.20</td>
<td>1453</td>
</tr>
<tr>
<td>Average</td>
<td>5.79</td>
<td>2014</td>
</tr>
<tr>
<td>CV %</td>
<td>17.2</td>
<td>30.8</td>
</tr>
</tbody>
</table>

Figure 9 – Typical failure exhibited by stack bond prisms after the compression tests: (a) front, (b) back and lateral sides.

3.3 Discussion

Figure 10 shows the comparison of the experimental results for prismatic and cylindrical samples, in terms of average values of Young’s modulus and compressive strength.

The values of the Young’s modulus of the investigated lime mortar masonry ranged between 1182 MPa and 2014 MPa, depending on the type of sample. The experimental Young’s modulus of stack bond prisms showed to be in good agreement with the values $E_2$ of core samples, i.e. when the stress was computed according to the cross-section of the regularized cap. The CV was only 6.6% in this case. Lower values were obtained for $E_1$ of core samples, i.e. when the stress related to the entire diametric cross-section of the core samples was considered in the calculations.

The experimental values of the compressive strength of masonry ranged between 3.98 MPa and 6.17 MPa, depending on the type of sample. The maximum strength was provided by $f_{c2}$ of the core samples 2JC. As for the core samples, the presence of the vertical mortar joint resulted in an average reduction of the compressive strength of -6.3%. However, this difference between the strengths of 2JCs and 3JCs actually depends
on the condition and integrity of the head joint. It is worth mentioning that the head joints are often badly executed in existing masonry, i.e. they may have been partially filled by the masons during the construction. For this reason, other experimental programs on masonry characterised by partially filled head joints might result in a higher discrepancy between the 2JC and 3JC compressive strengths.

It is worth mentioning that the calculation of the compressive strength $f_{C2}$ obtained from the tests on the cylindrical cores considering the width of the cap is much closer to the compressive strength of the prisms than $f_{C1}$, calculated considering the diameter of the cylinders. The coefficient of variation was only 3%.

![Graph](image)

Figure 10 – Comparison between compression tests on cylindrical samples and stack bond prisms: a) Young’s modulus, b) compressive strength.
The ratios between the experimental Young’s moduli and compressive strengths obtained for the different specimens were 278 for 2JCs, 332 for 3JCs, and 347 for stack bond prisms. All the obtained values are much lower than the ratios suggested by the current European and American standards, such as 1000 by CEN [31] and 700 by ACI [32]. This disagreement suggests that the reference ratios provided by these standards are not suitable for aerial lime mortar and brick masonry. On the other hand, the Italian standards [33] suggest typical ratios around 450 for masonry made of solid bricks and lime mortar.

4. Numerical simulation of compression tests on cylindrical samples

The extraction of cylindrical samples from existing masonry walls shows to be a very useful minor destructive technique for the evaluation of the compressive strength of existing masonry structures. Nevertheless, the presented experimental results illustrate some difficulties in the interpretation of the experimental results, such as the choice of the effective resistant area of the cylindrical samples, which is used for the determination of the compressive strength of aerial lime mortar and brick masonry. This section aims to provide a numerical insight into the evolution of the resisting and failure mechanisms of 2JCs and 3JCs.

4.1 Numerical models

The cylindrical samples are simulated by using a continuum finite element approach with distinct modelling of the mortar, the brick and the regularization mortar cap [22]. Figure 11 illustrates the used finite element meshes for the 2JCs and 3JCs. These are composed of 8208 isoparametric solid brick elements (9800 nodes) based on linear interpolation and $2 \times 2 \times 2$ Gauss integration. Only a quarter of the specimen is modelled, while proper symmetrical boundary conditions restraining only horizontal displacements are imposed to the interior surfaces of the specimen along the two vertical planes of symmetry to consider the effect of the non-modelled parts of the cylinders. The experiment is simulated by applying a vertical incremental displacement at the top of the regularization cap, precluding horizontal displacement, while the lower end is kept fixed. The system of nonlinear equilibrium equations is solved at each analysis step through the use of a secant method along with a line-search procedure. Convergence is attained when the ratio between the norm of the iterative residual forces and the norm of the total external forces
is lower than $10^{-2}$ (1%). The simulations are performed using the finite element analysis software COMET [34], while GiD [35] is used for the pre- and post-processing, both developed at the International Centre for Numerical Methods in Engineering (CIMNE) in Barcelona, Spain.

![Finite element meshes](image)

Figure 11 – Finite element meshes used in the numerical simulations for the two joint cylinder (left) and the three joint cylinder (right). The planes of symmetry are those having a normal vector with direction towards the $+x$ and the $-z$.

Table 6 presents the mechanical properties of the brick and mortar used in the numerical model. The tensile strength ($f_t$) for both materials was defined using the experimental results of the flexural tests and the expressions proposed by CEB-FIB [36]. The normalized uniaxial compressive strength of the bricks was measured through uniaxial compressive tests, presented in Table 2, and by applying the shape factor equal to 0.7 recommended by CEN [26]. The uniaxial compressive strength of the mortar was estimated through the DPT on mortar joints extracted from the dismantled wall of Figure 2a and presented in Table 3. The Young’s modulus of the units was measured in [15] by adopting the experimental procedure proposed for concrete by CEN [37]. The adopted value is the average given from cylinders extruded from the lateral sides of the brick (header and stretcher) and for cyclic loading up to 60% of unit’s maximum force [15]. The Young’s modulus of the mortar was defined such that the elastic stiffness of the
The numerical model was equivalent to that of the experimental one of the specimens 2JC4, 2JC13 and 2JC15 at the beginning of the experiment, which presented the highest stiffness. The formula suggested by the standard [36] has been used for the tensile fracture energy ($G_t$), while the compressive fracture energy ($G_c$) was the result of a sensitivity analysis aimed at obtaining a good agreement with the experimental results in the post peak nonlinear range. Linear elastic behaviour is assumed for the regularization cap with a Young’s modulus equal to 23000 MPa.

Table 6 – Mechanical properties of brick and mortar used in the numerical simulations

<table>
<thead>
<tr>
<th>Property</th>
<th>Brick</th>
<th>Mortar</th>
</tr>
</thead>
<tbody>
<tr>
<td>E (MPa)</td>
<td>7140</td>
<td>100</td>
</tr>
<tr>
<td>ν (−)</td>
<td>0.18</td>
<td>0.25</td>
</tr>
<tr>
<td>$f_t$ (MPa)</td>
<td>1.64</td>
<td>0.24</td>
</tr>
<tr>
<td>$f_c$ (MPa)</td>
<td>21.49</td>
<td>0.91</td>
</tr>
<tr>
<td>$G_{ft}$ (J/m$^2$)</td>
<td>126</td>
<td>80</td>
</tr>
<tr>
<td>$G_{fc}$ (J/m$^2$)</td>
<td>34400</td>
<td>1440</td>
</tr>
<tr>
<td>$f_{co}$ (MPa)</td>
<td>10.75</td>
<td>0.405</td>
</tr>
<tr>
<td>$\varepsilon_{pc}$ (−)</td>
<td>0.025</td>
<td>0.045</td>
</tr>
<tr>
<td>$f_{bc}/f_c$</td>
<td>1.15</td>
<td>1.5</td>
</tr>
<tr>
<td>$\rho$ (−)</td>
<td>0.75</td>
<td>0.75</td>
</tr>
</tbody>
</table>

The numerical analysis simulates the cracking and crushing of the mortar and bricks by using a continuum damage mechanics model with damage induced orthotropic behaviour along the principal axes [38]. The model uses two distinct damage indices to distinguish between tensile and compressive damage. The tensile response is characterised by a linear part until reaching the peak strength, and the post-peak part with an exponential softening law. For the compressive damage, the constitutive law presented in [39] is adopted, allowing for a parabolic hardening-softening part followed by exponential softening. Table 6 presents the selected values of the elastic limit of the compressive strength ($f_{co}$) and the strain ($\varepsilon_{pc}$) for the peak compressive strength ($f_c$) for each material. For both tensile and compressive laws, a residual strength equals to 10% of the maximum capacity is used. The adopted failure criterion is the one presented by Lubliner et al. [40]. The ratio between biaxial and uniaxial compressive strength ($f_{bc}/f_c$) is defined equal to 1.15 for the units and 1.50 for the mortar. Parameter $\rho$ defines the shape of the compressive failure surface under triaxial compression and is considered equal to 0.75 for both units and mortar. The latter values for $f_{bc}/f_c$ and $\rho$ are selected based on a sensitivity analysis due to the difficulty of their derivation from standard experimental procedures.
4.2 Numerical simulation of compression tests on two joint cylinders

Figure 12 presents the graphs of experimental and numerical force against vertical displacement at the top of the cap for the case of the 2JCs. For each experimental graph, the loading-unloading cycles are omitted and the part of the graph before them (not included in Figure 6b,d) is reproduced using the tangent to the curve after the end of the cycles.

![Graph of experimental and numerical force against displacement for 2JCs](image)

Figure 12 – Two joint cylinder specimens: graphs of experimental and numerical force against displacement at the top of the sample.

As said in the previous Section 5.1, the calibration of the Young’s modulus of the mortar has been based on specimens 2JC4, 2JC13 and 2JC15, which present the highest stiffness. For this reason, the stiffness and capacity predicted by the numerical simulation are very close to those given by the particular cylindrical specimens. It is noted that after the value of the mortar’s Young’s modulus was calibrated for the 2JCs, the same value was also used for the 3JCs, presented in the following Section 5.3. The numerical simulation predicts a peak strength of 103.0 kN. This value falls within the limits given by the experimental results, which are between 79.1 kN for 2JC13 and 107.9 kN for 2JC15.

The simulation shows that the first cracks in the cylinder appear close to the lateral edge of the regularization cap and affect mostly the upper and lower bricks and less the mortar joints. At this stage of the analysis, presented in the second column of Figure 13 (for a vertical displacement $u_y = 1.0$ mm), most compressive damage exists at the external parts of the mortar joints. The lower capacity of the external part of the mortar joints compared to the internal part is due to its different stress state. As shown in Figure 14, the external parts of the mortar joints are under a tension-compression stress state, while the internal parts are under triaxial compression allowing them to sustain higher compression. The
first drop of the capacity occurs for a vertical displacement at the top $u_y = 2.3$ mm and is associated to the opening of a crack at the middle of the central brick (third column of Figure 13). The compressive damage at the brick (Figure 13b) occurs under a tension-compression stress state. Compressive damage evolves also at the external parts of the mortar joints. After the appearance of the central crack, the cylinder recovers some of the capacity up to a vertical displacement $u_y = 2.5$ mm, when a second crack appears at the central brick (fourth column of Figure 13). The new crack results in the partial closure of the central one, as demonstrated from the contour of the maximum principal strains in Figure 13c. The rest of the analysis leads to the increase of the degradation of the mortar and the brick at the already damaged locations. When the tensile or compressive fracture energy is exhausted at one of these parts, another drop is obtained in the capacity curve (Figure 12).
Figure 13 – Numerical simulation of two joint cylinder specimens: evolution of (a) tensile damage, (b) compressive damage, (c) maximum principal strains $\varepsilon_{\text{max}}$ and (d) minimum principal strains $\varepsilon_{\text{min}}$ for different levels of vertical displacement.
Figure 14 – Numerical simulation of two joint cylinder specimens: vectors of the principal stresses at the mortar joints for a vertical displacement of $u_y = 1$ mm: (a) $\sigma_I$, (b) $\sigma_{II}$ and (c) $\sigma_{III}$.

Figure 15 presents the crack surface in the two joint cylinder at the end of the numerical simulation, as the isosurface of horizontal displacements of 1 mm. The numerical simulation predicts correctly the typical hourglass failure observed in the experimental results (Figure 7a).
The presented numerical analysis of the 2JCs has provided an insight into their resistant part during the compression tests, useful to understand how to evaluate their compressive strength. Figure 16 presents the distribution of the vertical stresses ($\sigma_{yy}$) at the middle of the mortar joint and the central brick for the 2JC in two instances of the numerical analysis. Before damage occurs to the central brick (Figure 16a), a similar stress profile is observed within both the mortar and the brick, with the vertical stresses increasing from the exterior to the interior and center of the cylinder. After the opening of the central crack, a stress redistribution occurs within the brick, affecting mostly the front part close to front face of the cylinder. This is because the central crack does not propagate through the whole length of the cylinder, as also observed in the fourth column of Figure 13. Regarding the mortar, the increase of the compressive damage at the external faces, reduces significantly the level of the vertical stresses at these locations. The interior part though, presents a plateau of high vertical stresses. Overall, the vertical stress distribution during the different stages of the simulation shows that the size of the regularization cap delimits the actual resisting area of the 2JC.

Figure 16 – Numerical simulation of two joint cylinder specimens: vertical stress distribution $\sigma_{yy}$ (a) before the opening of the central crack and (b) before the opening of the external crack at the central brick.

4.3 Numerical simulation of compression tests on three joint cylinders

Figure 17 presents the graphs of experimental and numerical force against vertical displacement at the top of the cylinder for the case of the 3JCs. As for the 2JCs, the part before the loading-unloading cycles is reproduced using the tangent to the curve after the
end of the cycles. The numerical analysis predicts a capacity of 96.7 kN for the 3JCs, which falls within the limits of the experimental results defined by the capacity of 3JC2 and 3JC8 samples of 74.6 kN and 104.9 kN, respectively. Similar to the experimental results, the numerical simulations predict a reduction of the capacity of the cylindrical cores due to the presence of the vertical joint of -5.9%. This value is very close to the experimentally measured reduction in the capacity of the cylinders of -6.3%, see Table 4 and Figure 10. The numerical simulation represents the ideal case of a perfectly filled head joint attached to the central brick, which is rarely the case of existing masonry. This fact justifies the slightly lower reduction of the capacity due to the existence of the head joint compared to the experimental results.

The first damage in the 3JC are the cracks at the top and bottom bricks close to their connection with the regularization mortar cap (second column of Figure 18a). Compared to the 2JC, the low tensile strength of the mortar leads to the appearance of the vertical crack at the central head joint for low levels of loading. This crack results in the gradual loss of the stiffness of the specimen until reaching the peak load for a vertical displacement at the top of the cylinder \( u_y = 3.1 \) mm. The drop of the capacity coincides with the appearance of another crack in the central brick, see third row of Figure 18. From that point on, compressive and tensile damage continues to increase in the already damaged locations, as can be seen in the last column of Figure 18. The second drop of the capacity, shown in Figure 17, coincides with the compressive failure of a part of the lower mortar joint.
Figure 18 – Numerical simulation of three joint cylinder specimens: evolution of (a) tensile damage, (b) compressive damage, (c) maximum principal strains $\varepsilon_{\text{max}}$ and (d) minimum principal strains $\varepsilon_{\text{min}}$ for different levels of vertical displacement.
The numerical simulation predicts also for the 3JC the typical hourglass failure as in the experimental results (Figure 7b). The surface of the lateral crack in the central brick at the end of the analysis and its propagation through the mortar joint is illustrated in Figure 19.

Figure 19 – Numerical simulation of three joint cylinder specimens: final failure pattern shown as the isosurface of horizontal displacements $u_x$ of 5 mm

Similar to the 2JC, Figure 20 presents the vertical stress distribution $\sigma_{yy}$ at the middle of the top bed joint and at the middle of the central brick and the head joint at two stages of the numerical simulation. The stress distribution is similar to that of the 2JC, with the exception of the lower stress gradients at the location of the head joint due to the early appearance of the central crack. During the whole length of the simulation, the parts of the cylinder outside the area defined by the regularization cap appear to have a limited contribution to the sustainment of the imposed vertical load.
Figure 20 – Numerical simulation of three joint cylinder specimens: vertical stress distribution $\sigma_{yy}$ (a) at an early stage of the analysis with the crack at the head joint already opened and (b) before the opening of the external crack at the central brick.

5. Conclusions

This research has presented an experimental and numerical investigation of the laboratory compression test of cylindrical samples core drilled from masonry walls. A specific type of traditional masonry has been considered, composed of handmade terracotta solid bricks and aerial lime mortar joints. Such material is frequent in existing construction, especially in the structures of the built heritage.

Two walls were built in the laboratory with the aim of reproducing the in-situ sampling and subsequent testing of masonry cores. One year after the construction, 150 mm diameter cylindrical specimens were extracted from the walls by using a dry core-drilling procedure (without water) in order to avoid disjointing of the samples. Some of the cores (2JC) presented two horizontal mortar joints, whereas some others had two horizontal mortar joints plus a vertical one (3JC). The cylindrical samples were regularized by using high strength mortar caps and then tested in the laboratory under compression. Stack bond prisms were also built and tested at the same time, in order to have a direct comparison between the novel tests on extracted cylindrical samples and the tests on prismatic samples conforming to the current standards for new construction.

The materials used in the present experimental campaign have presented low-to-moderate mechanical properties, with a normalized compressive strength of bricks of 21.5 MPa and a compressive strength of mortar of 1.63 MPa after 1 year of curing. The experimental tests on masonry samples have shown that a compressive strength above 4 MPa can be
obtained by using aerial lime mortar with proper carbonation level (mature period) and moderately strong units.

The average compressive strength of stack bond prisms has been 5.79 MPa, i.e. 3.6 times higher than the compressive strength of the mortar after one year of curing. The average Young’s modulus of stack bond prisms, of 2014 MPa, has provided a ratio to the compressive strength equal to 347.

As for the cylindrical samples, besides the 3JCs suggested by the UIC 778-3R guidelines [18], this research program has considered a type of cylindrical sample, identified as 2JCs, which does not include the vertical joint, in order to assess its effect on the measured strength. The 2JCs have provided an average compressive strength 6.3% higher than the 3JCs. However, this value may vary depending on the condition of integrity of head joints in the existing brickwork.

The compression test on 150 mm diameter cylinders has shown to be an appropriate technique for the evaluation of the compressive capacity of existing masonry. The hourglass type of failure has been observed in both 2JCs and 3JCs. The compressive strengths from core samples have resulted in a good agreement with those derived from the standard tests on stack bond prisms. However, the compressive strength of cylindrical specimens has to be calculated carefully. This work has evaluated it by making use of two extreme values, based on the assumptions of either a total diametric or a cap wide resisting cross-section [14,15]. The present study has shown that the second estimate (6.17 MPa for 2JCs and 5.78 MPa for 3JCs) provides compressive strength values more in agreement with those from stack bond prisms. The values of the Young’s modulus of the core samples have shown a good agreement with the results from the standard tests when, once again, the cap wide resisting cross-section is considered in calculation.

The paper has presented also the numerical simulation of the compression tests of the 2JCs and 3JCs with the aim to interpret their resisting mechanism and identify the procedure for the calculation of the compressive strength. The finite element micro-modelling technique has been considered for this purpose. The numerical simulations have predicted correctly the capacity given by the experimental results and the hourglass failure mechanism of both the 2JCs and 3JCs. The vertical stress distributions within the FEM models of the cylinders indicate that the parts outside the regularization parts have a limited contribution to the resistance of the specimen under compressive loading. The numerical study has complemented the experimental outcomes with further insight into
the mechanical behaviour of the extracted cylindrical specimens when tested under compression.

The research has presented an experimental and numerical understanding of a technique intended to assist experts working in the analysis of historical buildings, who are constantly in need for reference values for specific typologies of masonry and also require reliable validation criteria for the non-standard tests on samples extracted from existing structural members. Following this line, it is highly advisable to continue investigating and improving the experimental MDT procedures based on core drilling and testing of cylindrical samples, as they have shown to be very promising and minimally invasive. At the same time, the enlargement of the database of experimental results for other typologies of masonry materials is also suggested, in order to obtain more data allowing to validate and extend the prescriptions of the current technical standards applicable to existing masonry structures.

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Compliance with Ethical Standards

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Conflict of Interest: the authors declare that they have no conflict of interest.

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Highlights

• Construction and one year curing of aerial lime mortar and brick masonry walls
• Extraction of core samples from masonry walls to simulate MDT in-situ sampling
• Comparison between compressive parameters from core samples and stack bond prisms
• Numerical simulation by FEM micromodelling of compression testing of core samples
• Calculation of compressive parameters from cores considering cap’s cross-section