# IN-PLANE CYCLIC TESTING OF LOW-DENSITY AAC MASONRY WALLS

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## ABSTRACT

The main advantage of using autoclaved aerated concrete (AAC) blocks for masonry construction is their excellent thermal insulation capacity. This property increases with decreasing material density, which however is inversely proportional to the values of mechanical properties, including compressive strength. Hence, also considering the emerging issues on energy efficiency of buildings for which low density AAC masonry is greatly appreciated, the study of its structural performance becomes a relevant issue for the definition of suitable criteria and limitations for the design of safe masonry buildings, in particular for seismic design purposes. The experimental campaign presented in this paper aimed at investigating the seismic performance of low density AAC masonry. First, characterization tests on blocks, mortar and wallettes were carried out (vertical and diagonal compression tests). In-plane cyclic shear tests on six full-scale unreinforced low density AAC masonry walls were then performed with the aim to obtain a reliable description of the lateral cyclic behavior. Information regarding the displacement capacity, the correlation between experimental and analytical strengths and the dissipative behavior of masonry were derived. The results show a moderate displacement capacity of low density AAC masonry walls, strongly depending on the applied vertical load, and a good correlation between analytical and experimental lateral strengths. At the end of the shear cyclic tests, the residual vertical compression strength of some walls was also evaluated.

**KEYWORDS**: AAC masonry, experimental testing, seismic design

## INTRODUCTION

Continuously emerging issues on sustainability are rapidly being acknowledged by code requirements for construction materials and techniques for the realization of perimeter walls. For these reasons concepts related to the limitation of energy consumption and air-pollution emissions during the whole life-cycle and general sustainability requirements have to be accounted for in the selection of construction materials, as well as their thermal insulation properties may represent a key factor for several applications. Sustainability and building energy efficiency have hence become important issues in design and rehabilitation, they have to be considered together with structural requirements and they cannot be neglected even in seismic areas. Modern masonry structures allow excellent performances from the viewpoint of sustainability requirements [1, 2]. Among other materials adopted for the production of masonry units, Autoclaved Aerated Concrete (AAC) probably represents the one with the best sustainability performances. In fact, it is obtained from largely available materials (mostly silica sand), it does not require firing at high temperature (the curing process is carried in autoclaves at temperatures normally lower than 200° C) in the manufacture process (with multiple advantages including limited energy consumption and carbon dioxide emissions as well as mechanical properties characterized by a reduced dispersion) and its thermal insulation properties are so high that in many cases additional external insulation is not even necessary.

The thermal insulation capacity of AAC increases with the percentage of internal air bubbles forming during the expansion process, which also control the material density, with an inverse proportionality between the two characteristics [3]. In other words, as the percentage of air bubbles increases, thermal insulation properties are improved but material density decreases, with a general decrease of mechanical properties. The use of lighter units may hence cause a reduction of structural and seismic global performances, which can be partially mitigated by the tendency to adopt larger thickness walls for further increasing insulation. As regards the seismic performance, it should be also considered that the use of lower density units reduces the masonry self-weight and hence both vertical stress and seismic inertia forces. Other potential advantages of the use of AAC in seismic design may consist of its non-combustible and fire-resisting nature, since unfortunately fires are still common consequences of seismic damage. Therefore, the need for the assessment of the expected seismic performance of AAC masonry arose in several world countries and several experimental campaigns were carried out (e.g. [4,5]). Also based on the comprehensive testing activity carried out at the University of Texas at Austin [4], the TMS 402 [6] masonry design code includes a specific chapter devoted to the design of this masonry type.

The experimental campaign carried out at the European Centre for Training and Research in Earthquake Engineering (EUCENTRE, Pavia, Italy) and described in this paper was mainly motivated by the increasing interest regarding the use of low density AAC masonry in seismic areas. This requires however to find a compromise between minimum structural properties and other characteristics of this type of masonry, such as its high performance for what regards thermal insulation and, more in general, energy efficiency and sustainability. Some codes, such as for example Eurocode 8 [7] and the Italian building code [8] require indeed a minimum unit compressive strength of 5 MPa for structural use in seismic design, whilst they require a specific assessment for lower strength units. For these reasons, it is important to estimate the seismic performance of low density AAC masonry, in order to justify the use of this material in seismic areas. To this aim, the results of in-plane cyclic tests on six full-scale masonry piers with different applied loads, slenderness ratios and failure modes are presented.

## MATERIAL CHARACTERIZATION TESTS ON BLOCKS, MORTAR AND MASONRY TRIPLETS

Experimental tests were performed on AAC blocks to derive their main mechanical properties. They provided an average value of the dry density of 360 kg/m$^3$ and an average unit compressive strength in the vertical direction, $f_b$, (evaluated according to [9]) equal to 3.06 MPa. As mentioned before, the dry density, i.e. the density obtained by oven drying specimens until constant weight is reached, is the main characteristic identifying the AAC unit material.

Mortar mechanical properties were obtained by flexural and compression tests. Flexural tests allow to evaluate the tensile strength of the mortar specimens, while compression tests allow to derive the compressive strength of the two specimen parts obtained from the previous flexural tests. In accordance with [10], mortar specimens consisted of prisms of 160 x 40 x 40 mm, prepared in metallic molds (formworks). Once filled with mortar, the molds were sealed in polyethylene bags or in hermetic containers for the curing phase and tests were performed after different curing ages. Mortar specimens manufactured by EUCENTRE were tested after a curing time of 31 and 32 days and provided values of the average compressive strength equal to 10.50 and 10.22 MPa, respectively. Flexural and compression tests on mortar prisms were also carried out in the Xella Laboratory in Emstal (Germany), where specimens subjected to different curing times (7 days and 28 days) and temperature and humidity conditions (at 20°C and 65% of relative humidity and 100% of relative humidity in $H_2O$) were tested. Tests after a curing time of 7 days, with 65% humidity, provided an average compressive strength of 9.26 MPa, whilst tests after 28 days provided values of 15.75 MPa and 12.36 MPa, for the case of 65% humidity and 100% humidity, respectively.

The initial shear strength of masonry in the absence of vertical load was evaluated according to the procedures A and B reported in [11] on triplet and couplet specimens, respectively. Procedure A consists in subjecting each specimen to axial compression perpendicular to the mortar layers and applying a transversal load to the center of the specimen. At least three specimens have to be tested for each of the three precompression levels required by the standard. In this experimental campaign, relatively low compression levels were selected because of the low compressive strength of masonry units, i.e. 0.1 MPa, 0.2 MPa and 0.3 MPa. The average initial shear strength, $f_{vm0}$, derived by a linear regression to zero from the results obtained for the different compression levels, turned out to be equal to 0.29 MPa, while [12] suggests a characteristic value equal to 0.30 MPa. The average internal friction coefficient $\mu$, evaluated as the angular coefficient of the regression line, turned out to be equal to 0.5.

Procedure B consists instead in performing shear tests without any axial compression and provided an average value of the initial shear strength of 0.32 MPa, which is 10.3% higher than the value obtained by applying procedure A.

## VERTICAL COMPRESSION TESTS

Six 1000 x 1250 x 300 mm masonry specimens, identified with a progressive name from V1 to V6, were subjected to vertical compression tests, according to [13]. The wallettes were made of AAC units having dimensions of 500 x 250 x 300 mm and a nominal density equal to 350 kg/m$^3$. The test apparatus consisted of a hydraulic force-controlled press device, applying a monotonic or cyclic compression force on the masonry specimen uniformly distributed on the wall section. The deformation in the wall for increasing levels of load was measured by means of six displacement transducers..

Vertical compression tests allow to obtain some important characteristics of masonry. In the pseudo-elastic behavior range it is possible to derive information about masonry stiffness (Young's modulus) and Poisson's ratio. If the wall is tested up to failure, it is possible to evaluate the ultimate strength and obtain information about the deformation capacity in compression. The tests provided an average value of compressive strength $f_u$ equal to 1.91, with a standard deviation of 0.13 and a coefficient of variation of 0.07. The average vertical strain $\varepsilon_u$ corresponding to $f_u$, was equal to 1.60E-03, with a standard deviation of 1.12E-04 and a coefficient of variation of 0.07. The average value of the secant Young's modulus evaluated at a stress level equal to one third of the maximum vertical stress achieved during the test was equal to 1380.2 MPa, with a standard deviation of 199.41 and a coefficient of variation of 0.14. Finally, the average value of Poisson's ratio $\nu$ was equal to 0.17, with a standard deviation of 0.13 and a very high coefficient of variation of 0.76.

## DIAGONAL COMPRESSION TESTS

Six square masonry specimens of 1000 mm side and 300 mm thickness, identified with a progressive name from D1 to D6, were subjected to diagonal compression tests. The panels were made of 500 x 250 x 300 mm AAC units with a nominal density equal to 350 kg/m$^3$.

The load was applied using the same hydraulic press adopted for vertical compression tests. The specimen was positioned between two supports and inclined at 45° with respect to the horizontal direction and subjected to a compression force along its diagonal. The force was applied via steel shoes at the corners of the specimen. Displacements were measured by four potentiometers, placed along the diagonals.

Diagonal compression tests are aimed to evaluate the characteristics of shear stiffness and strength of masonry. The masonry diagonal tensile strength $f_t$ was calculated according to the criterion proposed by Frocht [14, 15], which considers the biaxial state of stress developing at the center of a homogeneous elastic specimen. The average value of $f_t$ turned out to be equal to 0.23 MPa, with a standard deviation of 0.03 and a coefficient of variation of 0.15. For each wallette, the shear deformation $\gamma_{av,el}$, computed at 1/3 of the maximum load, was evaluated as the sum of the absolute values of the average vertical and horizontal deformations measured by potentiometers. Its average value turned out to be 3.04E-04, with a standard deviation of 5.36E-05 and a coefficient of variation of 0.18. Finally, the average value of the shear modulus $G$ (secant at 1/3 of the failure load) was equal to 356.7 MPa, with a standard deviation of 15.94 and a coefficient of variation of 0.04.

## CYCLIC SHEAR-COMPRESSION TESTS ON AAC WALLS

Cyclic shear-compression tests were carried out to evaluate the in-plane behavior of unreinforced low density AAC masonry piers. Four walls were tested at the EUCENTRE Laboratory in Pavia, including two squat specimens (YTO01 and YTO02) with dimensions of 2500 x 2000 x 300 mm and two slender ones (YTO03 and YTO04) with dimensions of 1250 x 2500 x 300 mm. Every specimen was made of units with dimensions of 500 x 250 x 300 mm and a nominal density equal to 350 kg/m$^3$. Horizontal and vertical joints were filled with a thin layer mortar.

Fig. 1(a) shows the test setup adopted, consisting in a double-fixed system with a constant vertical load applied at the top by hydraulic jacks. The lateral load was applied using a horizontal displacement-controlled actuator and three cycles for each displacement level were performed. Specimens were built on a reinforced concrete foundation, fixed to the strong floor by means of post-tensioned steel bars. A reinforced concrete beam was placed at the top of each wall, with the aim of better distributing the forces transmitted to the masonry by the testing apparatus. A more detailed description of the adopted test setup and of the testing procedure can be found in [16].

The vertical loads applied, equal to 300 kN for specimen YTO01, 200 kN for YTO02, 150 kN for YTO03 and 100 kN for YTO04, represent relatively high vertical loads which can be realistically found in 2-3 story AAC masonry buildings. They correspond to values of the average compressive stress $\sigma_0$ equal to 0.4 MPa for specimens YTO01 and YTO03 and 0.27 MPa for panels YTO02 and YTO04.

In-plane cyclic tests were also carried out in the Xella Laboratory in Emstal (Germany) on two slender panels, YTO05 and YTO06, with the same geometrical characteristics of specimens YTO03 and YTO04. The test apparatus adopted (Fig. 1(b)) was similar to that used in Pavia. The horizontal actuator was connected to the center of the upper reinforced concrete beam. The two vertical actuators could only apply compression forces, hence limiting the combinations of vertical force and bending moment applicable at the top of the specimens.


**Figure 1: (a) Test setup of TREES Lab (EUCENTRE, Pavia); (b) test setup of the Xella Laboratory (Emstal); (c) Markers placed on specimen YTO01.**

Both specimen YTO05, tested under cantilever boundary conditions, and specimen YTO06, tested under a double-bending configuration, were subjected to a vertical load equal to 150 kN (corresponding to $\sigma_0$ = 0.4 MPa). The vertical load applied to specimen YTO06 was then increased to 200 kN ($\sigma_0$ = 0.53 MPa), to avoid a tension force condition in one of the two vertical actuators. With the only exception of test YTO06, in all cases the applied vertical load was maintained constant during the entire test.
Displacements were recorded by means of linear potentiometers. For the tests performed in Pavia, displacements were also measured through an optical acquisition system, by means of high-definition infrared cameras recording the coordinates of a grid of specific points on the specimen surface and following their evolution over time (an example for specimen YTO01 is reported in Fig. 1(c)).
Fig. 2 shows the final cracking pattern (left) and the horizontal force versus horizontal displacement curves (right) of each specimen. Notice that the scale of the horizontal axis is different for the different specimens, whilst the vertical one is kept constant. Different failure modes were observed, mainly depending on the specimen's geometry and applied vertical load. Shear failures were observed for the squat specimens YTO01 and YTO02, with a final cracking pattern characterized by diagonal cracks extending to the entire height of the walls. Specimens YTO03 and YTO04 showed an initial flexural behavior followed by a shear failure. The slender panel with a cantilever configuration (YTO05) exhibited a flexure-dominated behavior and at the end of the test the base section was entirely cracked. Specimen YTO06 failed according to a shear-dominated mechanism, with a diagonal crack along the whole height of the panel. The test on specimen YTO03 was interrupted at a drift level equal to 0.50% because of the appearance of a very large vertical crack extending to the entire height of the wall, also combined with a crushing failure of the upper right corner of the panel. All the other tests were stopped when a significant level of damage was reached (extensive cracking with potential danger for people and instrumentation). This condition occurred at nominal drift levels equal to 0.70% for specimens YTO01, YTO02, YTO04 and equal to 1.00% and 0.25% for panels YTO05 and YTO06, respectively. Observation of the force-displacement curves allows to confirm the failure mechanisms observed during the tests. As commonly observed in tests performed on unreinforced masonry walls (e.g. [16, 17, 18]), Shear failures are characterized by large hysteretic loops and by a significant strength and stiffness degradation. In case of mixed or

flexural failures, the experimental curves are thinner and less dissipative and shear strength degradation is not clearly observed.

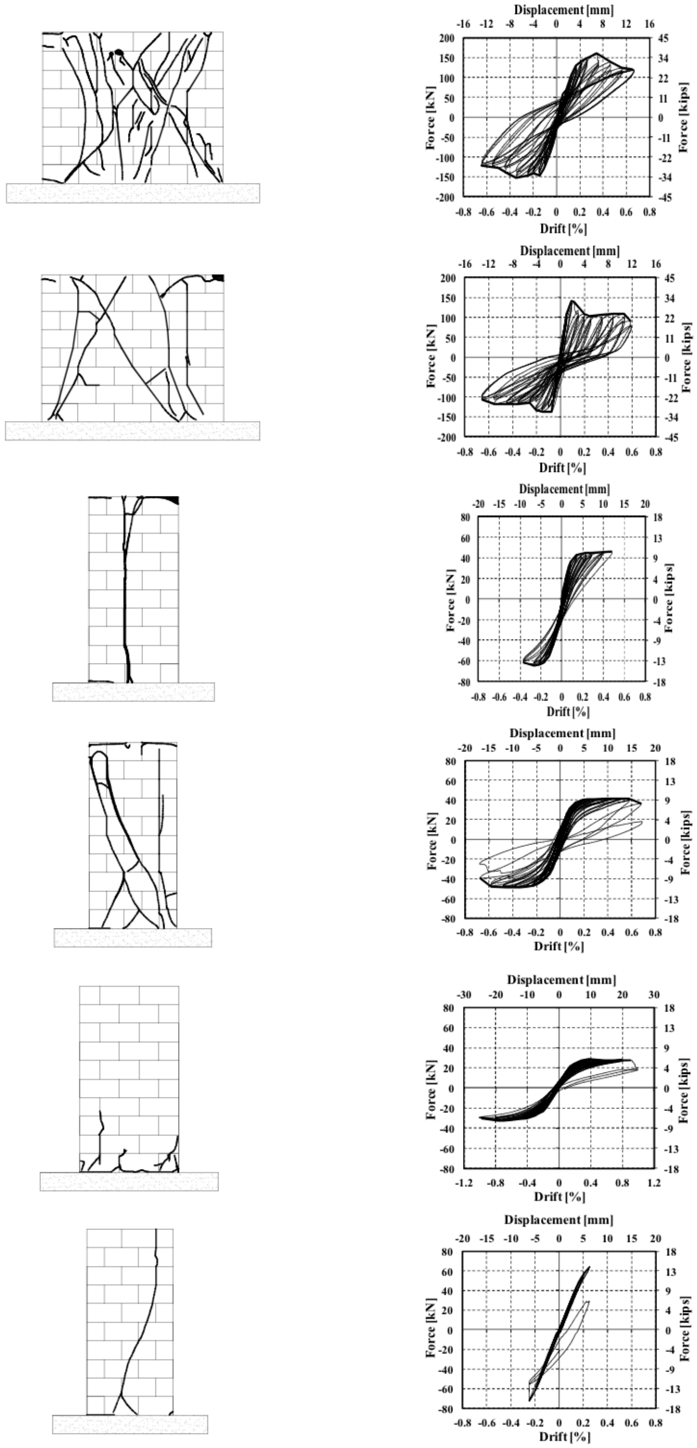


**Figure 2: Final cracking pattern (left) and horizontal force versus horizontal displacement curves (right) of specimens V1 to V6 (from top to bottom).**

After the in-plane cyclic tests, specimens YTO01 and YTO02 were subjected to a vertical compression test to evaluate the masonry residual compressive strength $f_{m,r}$, using the same test

apparatus adopted for the cyclic tests, with the vertical load applied by the two vertical actuators (the horizontal actuator was preliminarily disconnected). The masonry residual compressive strength, calculated as the ratio between the maximum vertical load applied during the test and the loaded area of the panel, turned out to be equal to 1.1 MPa for specimen YTO01 and 0.82 MPa for panel YTO02. The ratio between masonry residual compressive strength and (initial) masonry compressive strength $f_m$ (derived from characterization tests as previously described) was therefore equal to 0.57 and 0.43 for specimens YTO01 and YTO02, respectively.
The maximum strength exhibited by the tested walls was compared with the value determined as the minimum from the failure criteria suggested in the codes (e.g. [8], [12] and Annex C of [19]) for the evaluation of the flexural and shear strength, the latter associated with both diagonal cracking and sliding-shear mechanisms.
For the evaluation of the lateral strength of the masonry panels associated with bending failure, the strength criterion reported in [19] and [8] was followed. The shear formulation requires the definition of the masonry shear strength accounting for the presence of vertical load, for which both [8] and [12] propose a maximum value, $f_{v,lim}$, which is associated with the unit tensile failure, i.e. corresponding to diagonal cracking of the unit. However, as observed in [20], both the values of $f_{v,lim}$ proposed by [8] and [12] are not adequate for AAC blocks, because they were calibrated with reference to other masonry typologies. Therefore, $f_{v,lim}$ was evaluated by assuming $f_{v,lim} = \alpha f_b$, where $f_b$ is the unit mean compressive strength in the vertical direction. The coefficient $\alpha$ was calibrated based on the experimental values of lateral strength and it was found to be equal to 0.09 for the shear dominated squat specimens (YTO01 and YTO02) and equal to 0.19 in case of slender specimens with double-bending configuration (YTO03, YTO04 and YTO06). In the calculations a unique value equal to 0.09 was adopted.
The calculated values of maximum strength of the different specimens were consistent with the experimentally observed failure mode and the comparison of the calculated and experimental strength values for each specimen (Fig. 3) showed a generally good agreement, with a correlation coefficient equal to 0.83, indicating that the adopted formulation is quite accurate in predicting the experimental wall lateral strength.

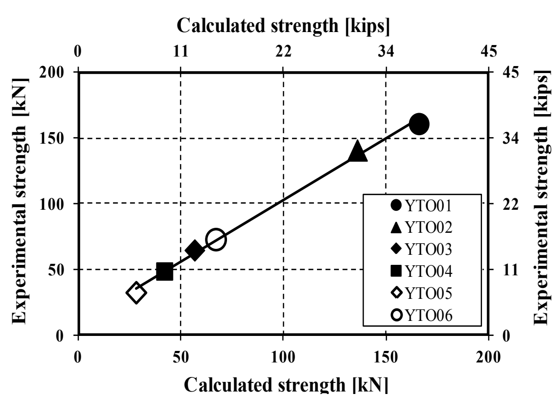


**Figure 3: Experimental versus calculated strength values.**

The displacement ductility capacity of each masonry panel can be estimated by approximating the cyclic test envelope curve with an equivalent elastic-perfectly plastic bilinear curve, which in this work was evaluated according to the same procedure adopted in [16], [20] and [21].In particular, the stiffness of the linear branch was defined as the secant stiffness to the point of the experimental curve where the shear force is equal to 70% of the maximum shear strength, $V_{max}$, whilst the ultimate displacement $\delta_u$ was defined as the displacement corresponding to a 20% decay of the maximum shear strength, or, in case a 20% drop of the maximum strength was not observed, as the displacement at which the test was stopped. In case the envelope curve showed a decrease in lateral strength higher than 20% but lower than 30% of maximum shear strength, followed by a stable almost constant branch, the ultimate displacement was computed as the displacement corresponding to the end of the almost constant branch. Finally, the ultimate strength $V_u$ was obtained according to the equal energy criterion, equating the areas below the experimental envelope and the bilinear curve and the yielding displacement $\delta_y$ was derived as the ratio between the ultimate strength $V_u$ and the effective stiffness $K_{eff}$. The displacement ductility $\mu_{eq}$ was obtained as the ratio between the ultimate and the yielding displacement. The parameters of the bilinear idealization curves obtained from the interpretation of the cyclic tests are summarized in Table 1, where the ultimate drift is the ratio between the ultimate displacement and the height of the specimen.

**Table 1: Summary of the parameters of the bilinear curves approximating the experimental cyclic envelop curves**

| Specimen | $\delta_y$ [mm] | $V_u$ [kN] | $K_{eff}$ [kN/mm] | $\delta_u$ [mm] | ultimate drift | $\mu_{eq}$ |
|---|---|---|---|---|---|---|
| YTO01 | 2.68 | 132.88 | 49.57 | 10.43 | 0.52% | 3.9 |
| YTO02 | 1.10 | 109.82 | 99.50 | 10.46 | 0.52% | 9.5 |
| YTO03 | 2.73 | 51.73 | 18.96 | 9.15 | 0.37% | 3.4 |
| YTO04 | 2.75 | 43.07 | 15.63 | 14.47 | 0.58% | 5.3 |
| YTO05 | 5.69 | 28.48 | 5.01 | 19.73 | 0.79% | 3.5 |
| YTO06 | 4.82 | 60.30 | 12.51 | 5.52 | 0.22% | 1.1 |

As reported in Figure 4, elaboration of the results shows that the ultimate drift values decrease almost linearly with increasing applied vertical compression stress .

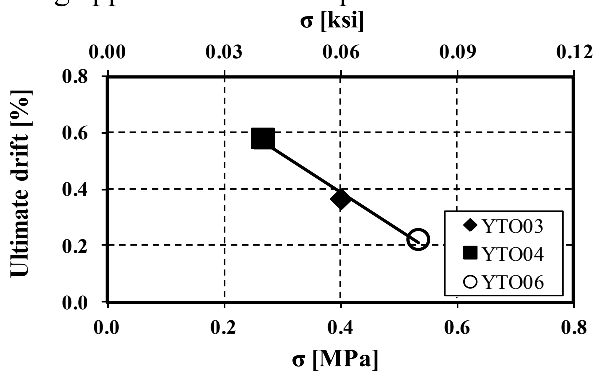


**Figure 4: Ultimate drift versus vertical compression stress.**

The ultimate strength of the bilinear curve, $V_u$, was compared with the maximum strength obtained from the experimental envelope curves, $V_{max}$, and the strength value corresponding to 70% of the maximum lateral strength, $V_{cr}$. The results are reported in Table 2, where it can be noted that the average value of the ratio $V_u/V_{max}$ was equal to 0.92, which is slightly higher than the value of 0.90 suggested by [22] and [23]. Similarly, the average value of the ratio $V_{cr}/ V_u$ was found to be 0.77, which is slightly higher than the value of 0.75 proposed by [22] and [24].

The values of effective stiffness $K_{eff}$ are reported in Figure 5. They are expressed as a percentage of the initial stiffness of the element $K$, calculated according to the classical formula taking into account both shear and flexural stiffness contributions.

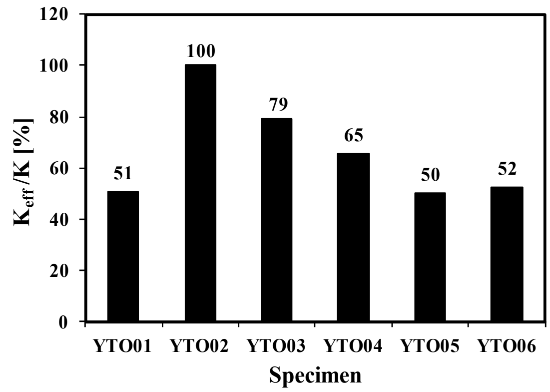


**Figure 5: Ratio of effective versus initial stiffness for the six specimens.**

Specimens YTO01, YTO05 and YTO06 provided a value of the ratio $K_{eff}/K$ of about 0.5. In the case of specimen YTO01, this value, relatively low with respect to what observed for the other squat wall (YTO02), could be partly due to some minor cracking of the specimen during the clamping of the foundation to the strong floor (higher effective stiffness would be expected in case of higher vertical compression, as it is the case for slender walls YTO03 and YTO04). The result obtained for walls YTO05 and YTO06 confirms the ratio of 0.5 typically used in the engineering practice, which is also suggested by some codes (e.g. [8]). Specimen YTO03 provided instead a value of the ratio $K_{eff}/K$ of about 0.8, which is in good agreement with the results obtained from previous experimental campaigns on AAC masonry piers with higher nominal densities (e.g. [20], [21]).

**Table 2: Summary of the parameters of the bilinear curves approximating the experimental cyclic envelop curves**

| Specimen | $V_{cr}$ [kN] | $V_u$ [kN] | $V_{max}$ [kN] | $V_u$ / $V_{max}$ | $V_{cr}$ / $V_u$ |
|---|---|---|---|---|---|
| YTO01 | 103.12 | 132.88 | 147.32 | 0.90 | 0.78 |
| YTO02 | 95.34 | 109.82 | 136.20 | 0.81 | 0.87 |
| YTO03 | 37.98 | 51.73 | 54.25 | 0.95 | 0.74 |
| YTO04 | 31.40 | 43.07 | 44.85 | 0.96 | 0.73 |
| YTO05 | 20.92 | 28.48 | 29.89 | 0.95 | 0.73 |
| YTO06 | 45.53 | 60.30 | 65.05 | 0.93 | 0.76 |
| Average | | | | 0.92 | 0.77 |

## CONCLUDING REMARKS

This paper summarizes the results of an experimental campaign carried out to evaluate the seismic performance of low density AAC masonry. This campaign included material characterization tests for the definition of the main mechanical properties of this type of masonry (standard tests on blocks, mortar and wallettes), followed by cyclic shear-compression tests on

six full-scale low density AAC masonry piers with different slenderness ratios, vertical loads and failure modes, performed to assess the in-plane behavior of this type of masonry.
The results of cyclic tests showed that the displacement capacity of low density AAC masonry decreases almost linearly with increasing values of the vertical compression stress. This suggests that appropriate limitations could be adopted in design and/or seismic analysis of AAC load-bearing masonry buildings. An appropriate displacement capacity should be set based on a consistent limitation of the design vertical loads acting on masonry piers.
In order to properly evaluate the shear strength of low-density AAC masonry piers, higher values of $f_{v,lim}$ with respect to those suggested by [12] and [8] seem to be adequate for low density AAC masonry. Based on the results of the experimental campaign presented in the paper, a value of the ratio between $f_{v,lim}$ and the unit mean compressive strength in the vertical direction $f_b$ equal to 0.09 could be potentially suggested for design.
The comparison between the experimental values of lateral strength and those calculated using the strength criteria reported in the codes showed that the correlation between analytical and experimental results is rather good and hence classical expressions are suitable to describe the lateral strength of low density AAC masonry, provided that an appropriate value is adopted for $f_{v,lim}$.

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