Enhancement of the seismic performance of AAC masonry by means of flat-truss bed-joint reinforcement

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SUMMARY:
Only few experimental tests are available in the literature on bed-joint reinforced load-bearing masonry. These are performed on prototypes with different masonry typologies, often reduced scale and, in any case, only for a small number of combinations of slenderness, axial load and boundary conditions. This work presents the results of an experimental campaign including in-plane cyclic tests on autoclaved aerated concrete (AAC) masonry panels with thin bed- and head-joints filled with glue-mortar. Some of the specimens are made of unreinforced masonry, whilst in other the masonry walls are reinforced by means of bed-joint flat-truss reinforcement only. Based on the results of these tests, complemented by specific tests performed on wallettes realised with the two different construction techniques, a possible strength criterion is proposed. The results indicate that the inclusion of bed-joint reinforcement has the double effect of improving masonry resistance and displacement capacity, hence reducing damage.

Keywords: horizontally reinforced masonry, AAC masonry, cyclic tests, displacement capacity

1. INTRODUCTION

The Autoclaved Aerated Concrete (AAC) is obtained from a mixture of cement, lime, water and sand that expands by adding aluminium powder. This reaction causes microscopic hydrogen bubbles to form, expanding the concrete to about two times its original volume. After evaporation of the hydrogen, aerated concrete is cut to size and form and it is transported to a large autoclave where, under a pressure of 12 bars and a temperature of about 190-200°C, the curing process is completed. Autoclaving is required to achieve the desired structural properties and dimensional stability. The result is a non-organic, airtight, non-combustible, fire-resisting material characterised by its fine cellular structure, with diameter of air pores ranging from 0.1 to 2 mm.

The reinforcement used in the experimental campaign is made of two parallel galvanised steel thin plates connected by a continuous steel wire welded to the parallel bars in a truss-like fashion (see Fig. 1.1).
The main advantages of truss reinforcement in horizontally reinforced masonry (HRM) can be enumerated as follows:

- The truss-like element provides lateral confinement to the masonry bed-joint across its width (wall thickness);
- The reinforcement provides resistance to out-of-plane deformation of the masonry wall;
- Bed-joint thickness is considerably contained. When special mortars are used to reduce joint thickness, the use of flat-truss reinforcement is advantageous because it does not significantly increase the joint thickness;
- The welded connections provide a considerable degree of anchorage, which is comparable to that offered by anchoring hooks in normal reinforcement bars;

- The presence of horizontal reinforcement is known to confer increased deformation and ductility capacity to the horizontally reinforced masonry wall.

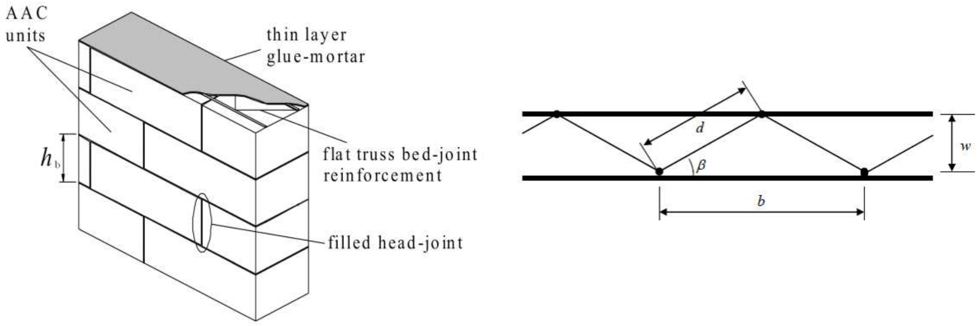


Figure 1.1 Truss type reinforcement placed in the horizontal bed-joint of a AAC masonry wall (left); truss type reinforcement with welded joints (right)

As reported in Penna *et al.* (2010b), several literature works studied the effectiveness of different types of bed-joint reinforcement in masonry walls, either along with vertical steel (e.g. Priestley and Bridgeman, 1974; Schultz *et al.*, 1998; Mosele *et al.*, 2009) or within confined masonry walls (e.g. Ganz and Thürlimann, 1984; Aguilar *et al.*,1996; Alcocer and Zepeda, 1999; Lourenço *et al.*, 2008; Penna *et al.*, 2008). Nevertheless, the amount of data from tests on masonry walls with bed-joint reinforcement only, which is the focus of the present work, is rather limited (e.g. Tomaževič and Žarnić, 1984,1985; Lourenço *et al.*, 2008; Penna *et al.*, 2008; Mosele *et al.*, 2009). Also, most of these tests were carried out on scaled wallettes and only two of these works included specimens with truss-type reinforcement (Lourenço *et al.*, 2008; Penna *et al.*, 2008).
In particular, only one of the test specimens of the experimental campaign performed at the EUCENTRE of Pavia (Penna *et al.*, 2008) exactly fits the typology investigated in the this study (i.e. only truss-type reinforcement in AAC masonry), although within the same campaign two other AAC walls with bed-joint reinforcement only provided comparable results. Indeed, the presence of horizontal reinforcement only induced an enhancement of lateral capacity increasing shear strength. The shear the failure modes observed in unreinforced AAC masonry walls tested under same conditions of aspect ratio, vertical compression and boundary conditions, became all bending failures in bed-joint reinforced specimens and the displacement capacity associated with bending failure modes in AAC URM walls was also largely increased.
For the reasons outlined above, the present study aims at investigating in details the effect of flat-truss bed-joint reinforcement on the lateral shear behaviour of AAC masonry walls.

2. EXPERIMENTAL TEST CAMPAIGN ON LOAD-BEARING AAC WALLS

2.1. Test specification

An experimental test campaign was performed to assess the in-plane shear behaviour of AAC masonry piers and to investigate the effect of flat-truss bed-joint reinforcement. The test setup (shown in Fig. 2.1) consisted in a double fixed system with a constant vertical load applied at the top by servo-controlled hydraulic actuators. The lateral load was applied in terms of increasing displacement using a horizontal actuator. Three cycles for each displacement level were performed.

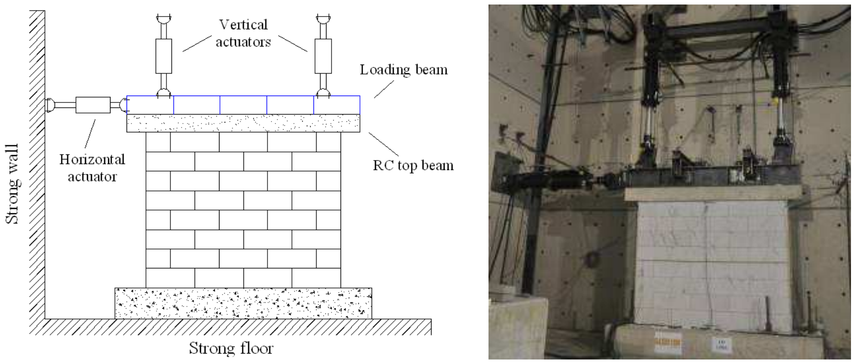


Figure 2.1 Scheme (left) and view (right) of the test setup

One unreinforced masonry (URM) wall and two horizontally reinforced masonry walls with truss type reinforcement (HRM) have been tested. All the masonry piers have the same geometry with a length (*l*), height (*h*) and thickness (*s*) of 2.5 m, 2 m and 0.3 m, respectively. Both bed-joints and head-joints were filled with a thin layer of glue-mortar. The dimensions of the blocks are 625 x 300 x 250 mm^3. The flat-truss bed-joint reinforcement is placed in the wall's height every two courses (50 cm), for a total of four horizontally reinforced layers.

With the aim of better distributing the applied loads and reproducing a typical constructive detail, a reinforced concrete beam was built at the top of each wall. The mechanical properties of masonry were measured experimentally by specific tests, performed on blocks, glue-mortar, wallettes and horizontal reinforcement, according to the EN 1015-11, EN 771-4, EN 772-1, EN 1052-1, EN 1052-3 and EN ISO 15630-2 standards. Table 2.1 summaries the values obtained from the tests for the most relevant parameters: f_b and f_m are the block and masonry compressive strength in the direction perpendicular to the horizontal mortar layers, $f_{v,lim}$= 0.1f_b is the maximum value of the masonry shear strength (the coefficient 0.1 is derived from the interpretation of the experimental results), f_{vk0} and f_{vm0} are the characteristic and mean initial shear strength in the absence of compression, ρ_m is the masonry density, A_{sw} is the area of the cross section of the bed-joint reinforcement and f_{yk} is the characteristic steel yield strength (provided by the producer). The values of the shear modulus *G* in Table 2.1 are calculated from Young's modulus *E* by assuming the empirical relationship *G*=0.26*E* (Costa *et al.*, 2011) and *G*=0.32*E* (obtained according to the experimental results), for the unreinforced and horizontally reinforced masonry, respectively.

Table 2.1. Experimental test results on masonry components

	f_b [MPa]	f_m [MPa]	f_{vk0} [MPa]	f_{vm0} [MPa]	E [MPa]	G [MPa]	ρ_m [kg/m^3]	A_{sw} [mm^2]	f_{yk} [MPa]
URM	3.48	2.33	0.17	0.25	1400	364	500	-	-
HRM	3.48	2.63	0.17	0.25	1400	448	500	24	600

Two levels of vertical load were applied: 300 kN (corresponding to an average compression of 0.4 MPa) for the unreinforced wall (RDB01) and one of the horizontally reinforced walls (RDB02), 450 kN (0.6 MPa) for the other reinforced wall (RDB03). These values correspond to an axial load ratio of 0.15 and 0.23, respectively. The axial deformation of the horizontal reinforcement was evaluated by strain gauges (4 for RDB02 and 6 for RDB03, respectively) fixed on the truss-like elements. Displacements and possible slipping were measured by 23 linear potentiometers installed on the panels.

2.2. Interpretation of the results

Figure 2.2, Figure 2.3 and Figure 2.4 show the shear-displacement diagrams and the final cracking patterns of the three tested specimens. All panels showed a typical shear failure behaviour, with a response characterized by wide hysteresis loops and degradation of stiffness and strength, as evident from the figures. Considering the envelope of the cycles, there is a short initial linear trend followed by a descending branch associated with the spread of cross-diagonal cracks. The final patterns show diagonal cracks mainly affecting the blocks and crossing the centre of the walls. In the horizontally reinforced specimens, the most significant damage is located in the unreinforced joints, as the presence of the reinforcement limits the spreading of cracks which are larger and more numerous next to the unreinforced joints.

The ultimate and maximum displacement capacity of the specimens were expressed in terms of drift ratios (δ_u and δ_{max}, respectively, defined as the ratio between the horizontal displacement and the height of the wall). The ultimate drift ratio is defined as that corresponding to a lateral strength decay of 20% (i.e. using the displacement corresponding to a post-peak 20% decay in the shear-displacement curve). Values of δ_u of about 0.65% and 0.8% were found for the unreinforced and both the horizontally reinforced panels, respectively. All the tests were actually stopped beyond this limit when the tested specimens showed extensive damage with potential danger to the testing apparatus. This condition occurred for levels of drift equal to 0.7%, 1.25% and 1%, respectively for RDB01, RDB02 and RDB03.

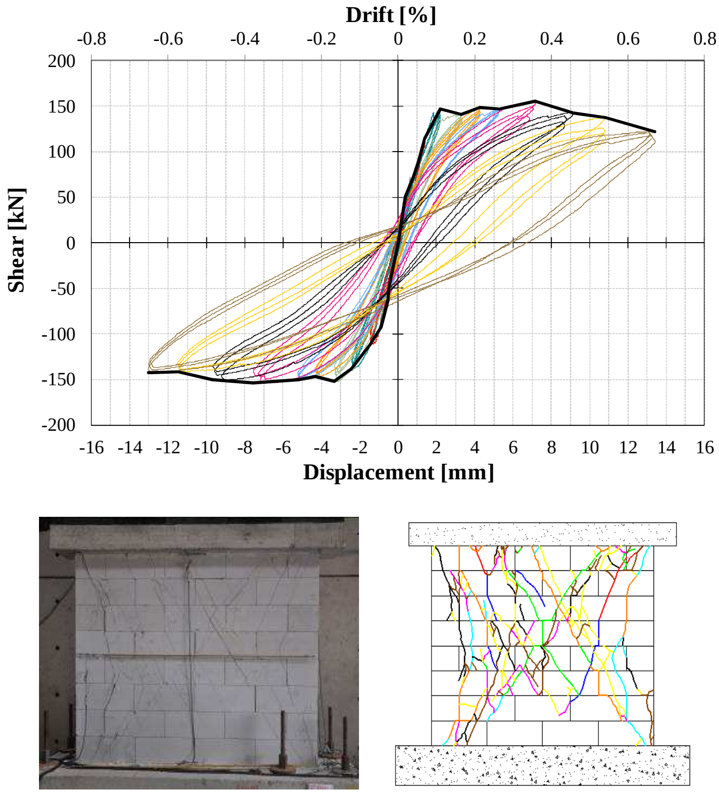


Figure 2.2 Wall RDB01. Top: hysteretic loops and force-displacement envelope (black thick curve). Bottom: view (left) and scheme (right) of the final cracking pattern (colours correspond to different drift's cycles)

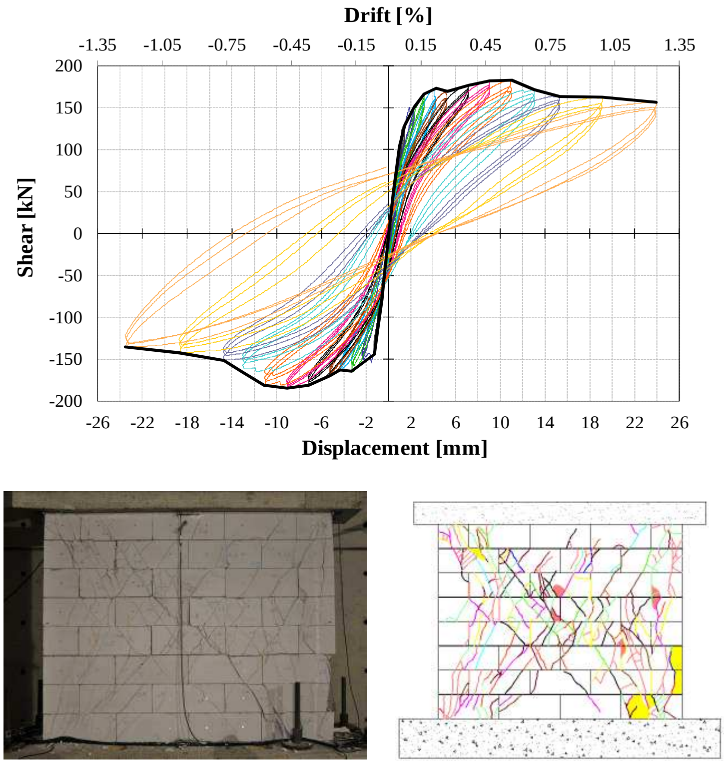


Figure 2.3 Wall RDB02. Top: hysteretic loops and force-displacement envelope (black thick curve). Bottom: view (left) and scheme (right) of the final cracking pattern (colours correspond to different drift's cycles)

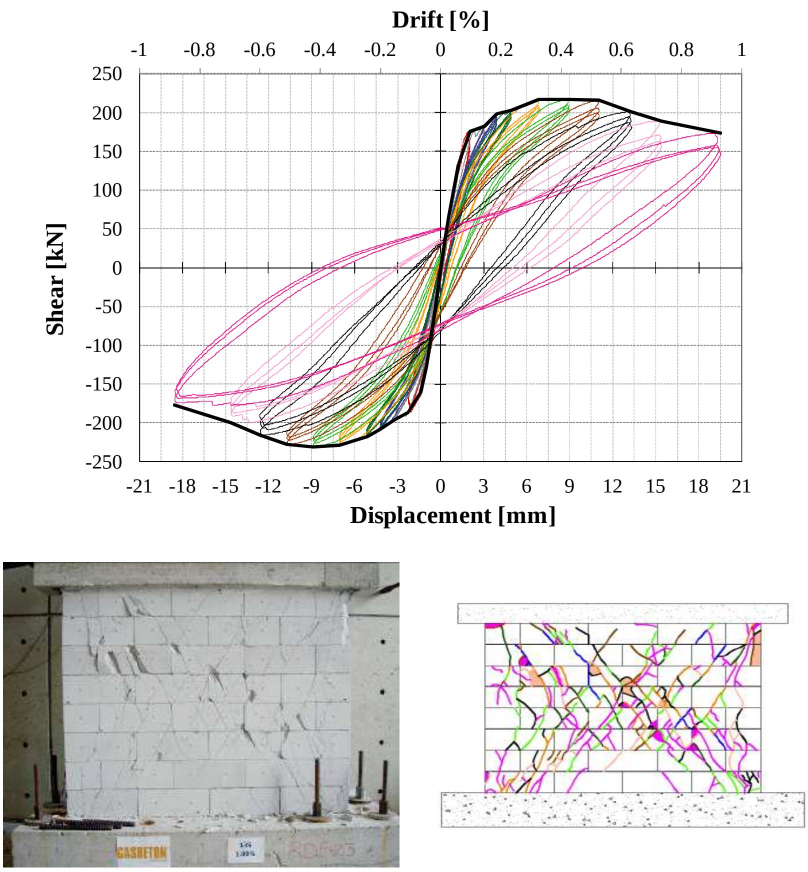


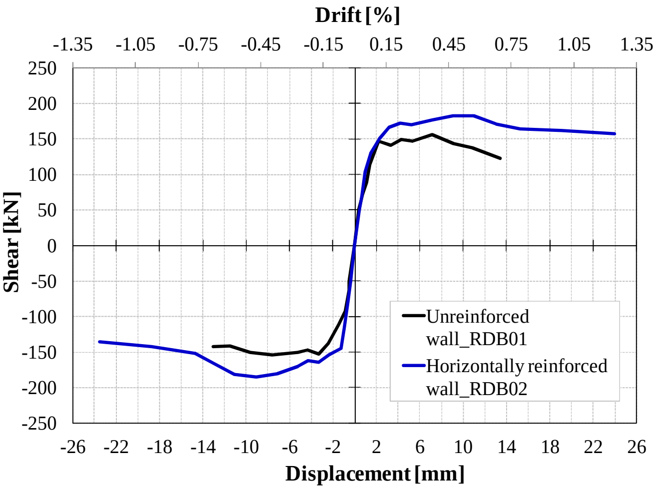
Figure 2.4 Wall RDB03. Top: hysteretic loops and force-displacement envelopes (black thick curve). Bottom: view (left) and scheme (right) of the final cracking pattern (colours correspond to different drift's cycles)

Table 2.2 summarises the failure mechanisms, the maximum shear strength (V_{max}) and the ultimate drift levels (δ_u) for each test.

Table 2.2. Summary of the principal experimental test results on masonry walls

Specimen	Failure mechanism	V_{max} [kN]	δ_u [%]
RDB01	Shear	156	0.65
RDB02	Shear	185	0.82
RDB03	Shear	231	0.86

Figure 2.5 reports a comparison between the envelopes of the three shear-displacement curves. It can be noted that the horizontally reinforced specimen shows an increase in the deformation capacity of approximately 80% compared to the unreinforced solution, while the increase of the shear strength is more contained and it can be estimated equal to 15-20%. The initial stiffness in almost identical in the three cases. As expected, the increase of vertical load induced a reduction of the horizontal displacement capacity and an increase of the strength, both of the order of 15-20%.



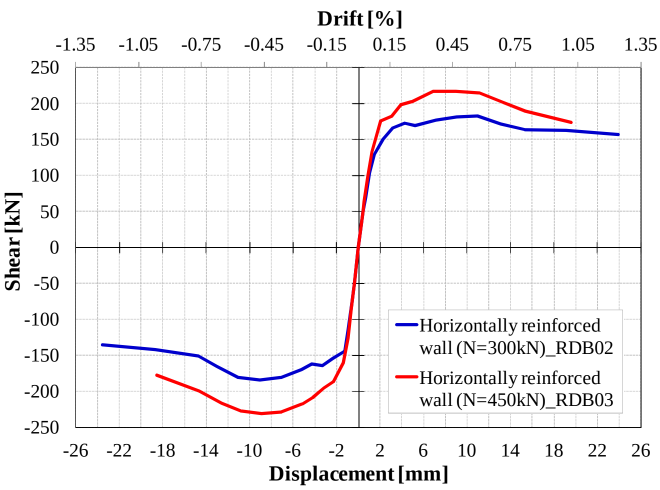


Figure 2.5 Comparison of the force-displacement envelopes. Top: unreinforced (RDB01) and horizontally reinforced case (RDB02), same vertical axial load. Bottom: horizontally reinforced walls with different vertical axial load (RDB02 and RDB03)

A bilinear approximation of the shear-displacement envelopes of the various tests was constructed, according to NTC08 (and in particular its Commentary, MIT, 2009) and EN 1998-3 Annex B (as far as applicable). The definition of equivalent bilinear elastic-perfectly plastic curves requires identification of a displacement range, from zero to ultimate displacement (Δ_u), the ultimate strength (V_u), the stiffness of the equivalent linear branch (k_y) and the yielding displacement (Δ_y). The ultimate displacement was assumed as the displacement corresponding to a drop of lateral resistance equal to 20% of the maximum experimental strength (V_{max}). The stiffness of the first branch was determined as the secant stiffness to the point of the capacity curve where the lateral resistance is equal to 70% of the maximum strength. The ultimate strength was found according to the equal energy criterion, by setting the total area below the bilinear curve equal to the same value of the experimental one. Finally, the remaining parameter (Δ_y) was easily derived from the ratio between V_u and k_y.

This procedure allows also to evaluate an apparent available displacement ductility (μ_D), defined as the ratio between the ultimate displacement (Δ_u) and the yield displacement (Δ_y).

The values of the parameters of the bilinear approximation of the shear-displacement envelopes (maximum in absolute value) are summarized in Table 2.3, in which k_t is the theoretical stiffness (calculated assuming the elastic moduli of Table 2.1 and the elastic Timoshenko beam model). It could be noticed that, as expected, the ductility capacity exhibited by the horizontally reinforced walls is higher than that obtained for the unreinforced one.

Table 2.3. Parameters of the equivalent bilinear approximation to the shear-displacement envelopes

Specimen	V_{max} [kN]	V_u [kN]	Δ_y [mm]	Δ_u [mm]	Δ_{max} [mm]	k_y/k_t [-]	μ_D [-]
RDB01	156	144	1.77	13	13.41	0.81	7.36
RDB02	185	169	1.46	16.3	23.94	0.96	11.13
RDB03	231	231	1.86	17.2	19.54	0.96	9.22

The comparison between the energy dissipated by hysteresis during the three tests shows that the presence of horizontal reinforcement, in addition to containing the spread of cracks, also increases considerably the final dissipation capacity of masonry piers (see Fig. 2.6), although the dissipated energy is generally lower for corresponding displacements due to damage limitation.

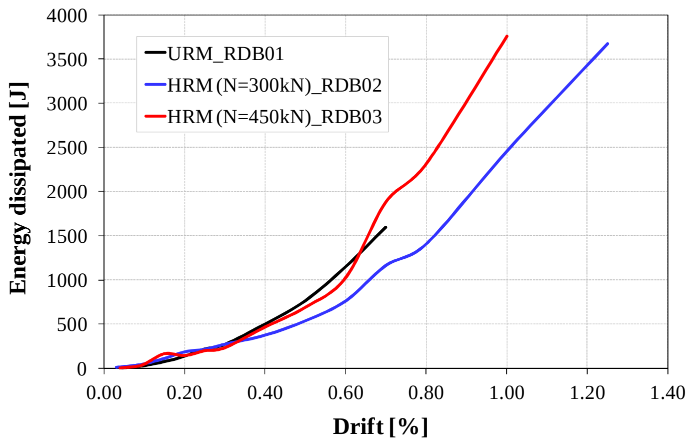


Figure 2.6 Comparison of the energy dissipated by hysteresis

An inspection conducted on the panels after the tests allowed the identification of several fracture points in the reinforcement located in the central horizontal joints. As evident from Figure 2.7, the damage was aligned with the main systems of diagonal cracks formed in the specimen.

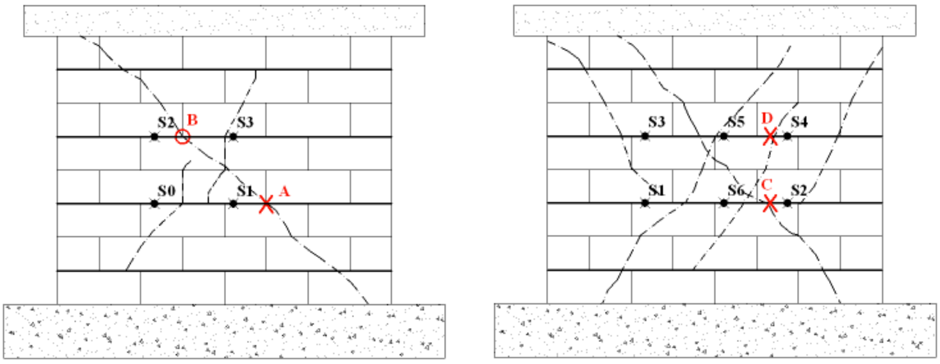


Figure 2.7 Pattern of the failures (red points) of the horizontal reinforcement in the specimens RDB02 (left) and RDB03 (right)

Figure 2.8 shows that these failures of the reinforcement (red points in Fig. 2.7) affected the entire cross section of the truss reinforcement and were also detected by the strain gauge measurements which, during the test, recorded sudden jumps in the horizontal reinforcement deformation, although the fracture was positioned at some distance from the gauge. The sound of the fractured steel could also be heard during the tests.

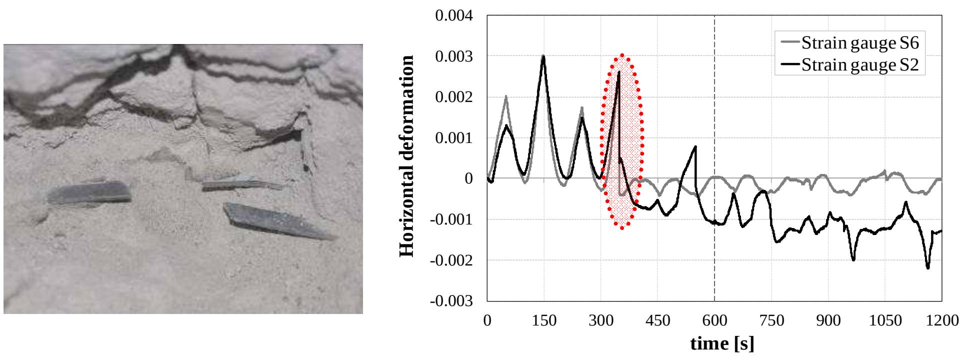


Figure 2.8 Picture of the failures of the horizontal reinforcement (left) and horizontal deformation recorded by the strain gauges (right)

3. PROPOSED CRITERION FOR SHEAR STRENGTH OF HORIZONTALLY REINFORCED AAC MASONRY

The increase of shear resistance offered by flat-truss bed-joint reinforcement was estimated using the criterion of resistance reported in Eqn. 3.1, which is an improvement of the formulation initially proposed, for horizontally reinforced AAC masonry, by Penna *et al.* (2007):

$$V_{t,H} = V_{sl} + V_{R,H} = f_v l' t + \alpha A_{sw} f_y \frac{d'}{s} \leq f_v l t \tag{3.1}$$

where f_v is the masonry shear strength, l is the wall length, t is the wall thickness, l' is the length of the compressed portion of the wall, f_y is the steel yield strength, A_{sw} is the section area of the flat-truss reinforcement, s is the vertical spacing of the bed-joint reinforcement, h is the wall height, d'=min(l',h) and α is a coefficient of efficiency of the horizontal reinforcement.

In Eqn. 3.1, the shear resistance of the horizontally reinforced elements is obtained as the sum of the contribution of the unreinforced wall, V_{sl}, calculated according to the classical criteria of resistance (e.g. Costa *et al.*, 2011), and a contribution due to the horizontal reinforcement alone ($V_{R,H}$). A value of the coefficient α equal to 0.85 was identified as the one giving the best-fit with the experimental results in terms of shear strength, obtained for the AAC walls reported in this work .

Figure 3.1 reports the interaction diagrams for the three tested walls, with the curve of maximum shear versus axial load for each failure mode. It is noted that the diagram for the horizontally reinforced specimens (black line) was calculated using the proposed criterion of Eq. (3.1) and, with a calibrated efficiency coefficient, it fits rather well the experimental results, as indicated in the figure.

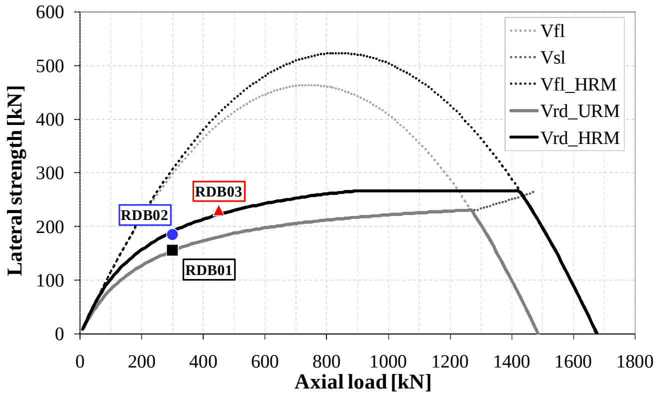


Figure 3.1 Interaction diagram of the tested walls (double fixed): unreinforced (grey thick curve) and horizontally reinforced masonry (black thick curve)

4. COMMENTS AND CONCLUSIONS

The paper summarised and discussed the results of cyclic shear-compression tests aimed at characterizing the seismic response of a new system of autoclaved aerated concrete masonry reinforced with only horizontal reinforcement placed in thin horizontal joints filled with glue-mortar. The three panels tested showed a typical shear behaviour with diagonal cracks mainly affecting the blocks and crossing the centre of the panels. In horizontally reinforced specimens the most significant damage was located in the unreinforced horizontal joints.

The presence of horizontal reinforcement (every 50 cm) allowed an increase of the deformation capacity of the wall with respect to the unreinforced solution. For what concerns the shear strength of the walls, the increase was equal to about 15-20%. Compared to the test with lower axial load, the increase of vertical load (specimen RDB03) generated a reduced horizontal deformation capacity and an increase in resistance roughly of the same order of magnitude (15-20%).

The maximum ultimate drift ratio associated with the shear failure mechanism increased, compared to the values proposed in the literature for unreinforced AAC masonry (Ötes and Löring, 2003; Tanner *et al.* 2005; Penna *et al.* 2008; Costa *et al.* 2011), from 0.3-0.35% to 0.8%.

This study has also led to the proposal of a strength criterion for AAC masonry with only horizontal reinforcement which needs however to be further validated with additional testing. Despite the limited number of tests that were carried out, the research confirms the effectiveness of the horizontal reinforcement in increasing the seismic performance of the walls, especially as a result of a significant enhancement of their deformation capacity and reduction of the damage. Extensive testing activity on bed-joint reinforced masonry would be necessary to better clarify the actual contribution to the shear resistance offered by the sole presence of horizontal reinforcement.

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