

DESIGN OF MASONRY STRUCTURES WITH BED JOINT REINFORCEMENT



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ABSTRACT

Bed joint reinforcement is a easy solution to improve significantly the structural performance of masonry walls, both for in-plane and out-of-plane response. The experimental tests summarised in the paper show the increment of lateral strength and displacement capacity, as well as energy dissipation, of both horizontally reinforced in-fill panels and masonry bearing walls. The comparison with other reinforcement techniques shows relevant advantages of bed joint reinforcement. Design tools have been developed for bed joint reinforcement made of prefabricated steel trusses and strength criteria are proposed for masonry piers with only horizontal reinforcement.

1. INTRODUCTION

The use of bed joint reinforcements in brick or block masonry is a easy and intuitive method which has been traditionally used and suggested in many reinforced masonry solutions, both for infill panels and bearing walls.

Among others, the benefit of such technical improvement are both structural and practical:

Bed joint reinforcements can be easily placed in the horizontal mortar layers, without any significant modification to the construction scheme;

Rebar horizontal reinforcement can be lodged within the thickness of traditional mortar layers (1 – 2 cm) or, in case of thin mortar layers (1.5-3 mm), they have to be combined with special grooved blocks;

Truss reinforcement improve the structural effectiveness of bed joint reinforcement both reducing anchoring lengths and allowing the transversal connection of double veneer walls;

Rounded truss reinforcement can be used in masonry walls with traditional mortar layers and flat truss reinforcement in thin mortar layers;

The presence of even slight horizontal reinforcement is very effective in cracking control;

Masonry spandrel beams and lintels can be easily built using bed joint reinforcements which provide flexural strength;

As shown in the following strength, displacement capacity and energy dissipation can be significantly enhanced in horizontally reinforced masonry piers, with an important improvement of the seismic response, in particular;

Horizontal slight reinforcement of infill panels provides damage reduction and enhancement of both in plane and out of plane lateral capacity, with possible beneficial effects also on the overall structural behaviour to wind and earthquake loading.

Although these technology has been planned for new constructions and typically it is carried out using steel reinforcement, various attempts have been done in order to obtain feasible retrofitting techniques which benefit from the use of horizontal reinforcements, but, even using stainless steel of other materials which are protected against corrosion, no generally applicable methods have been developed

2. IN-FILL MASONRY WALLS WITH BED JOINT REINFORCEMENT

As suggest first by Klingner and Bertero [5] and Brokken and Bertero [2], who have investigated the behaviour reinforced infill panels, slight reinforcements can provide superior performance, in terms of strength, stiffness, and energy dissipation, compared with a bare frame.

Starting from suggestions Calvi and Bolognini [3] have performed researches oriented to compare the in plane response of traditional and slightly reinforced hollow masonry infill panels, for different intensity level earthquakes, in order to assess the potentially different damage level attained; to assess the potential for out of plane expulsion of traditional and slightly reinforced masonry infill panels, at different level of damage induced by in plane action; to evaluate the effects of different properties of the infill panels on the response of buildings with different geometrical configurations and different distribution of the infill panels, in terms of PGA required to induce given level of damage.

Penna *et al.* [9] have then extended these experimental results in order to assess their value to a different kind of infill walls, made of aerated autoclaved concrete blocks, with a significant thickness (30 cm) and with thin mortar layers.

2.1. Hollow clay bricks with traditional bed joints

The main test campaign was performed on one-bay, one-storey full scale infilled frames, considering one single geometry and one single type of clay unit, for obvious budget reasons. Three possible reinforcement conditions were compared: no reinforcement, reinforcement in the mortar layers at 600 mm distance and light wire meshes in the external plaster. Interstorey drifts equal to 0.1, 0.2, 0.3 and 0.4 % were considered relevant for serviceability – low damage limits states and drifts equal to 1.2 and 3.6 % were considered significant to explore heavy damage or close to collapse limit states. At different levels of in plane drifts, out of plane tests were performed to define strength domains as a function of in plane damage.

The clay blocks were selected as typical highly perforated units used in the European earthquake-prone countries, with the following properties.

The horizontal reinforcement used for the mortar layers consisted of either two 6 mm re-bars with 578 MPa yield strength, 638 MPa ultimate strength and 24 % elongation capacity measured on 5 diameters, or by a welded truss made with two 5 mm bars with 630 MPa yield strength, 690 MPa ultimate strength and 16.5 % elongation capacity measured on 5 diameters (10.5 % on ten diameters, 3.34 uniform at maximum stress).

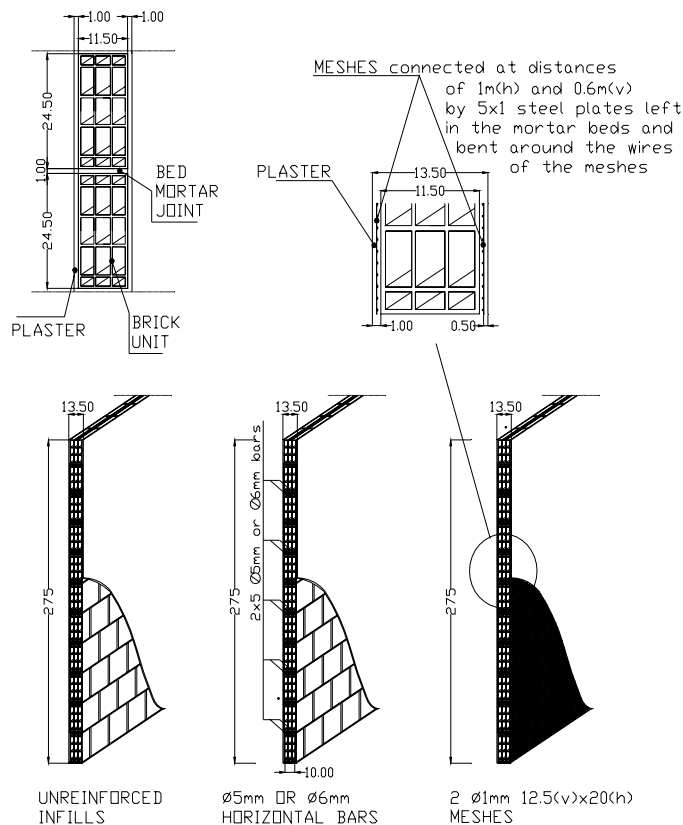


Figure 1: Details of the position of the steel bars and of the meshes to reinforce masonry panels (after [3])

As already pointed out, a single geometry was considered for the main test campaign on full scale infilled frames. The overall dimensions were selected to be 4.5×3 (height) m. The concrete frame was designed as the lowest part of a four storey building, applying thoroughly the rules given by Eurocode 2 and Eurocode 8. The structure was designed according to the high ductility class of EC8, applying fully the capacity design rules. The plastic hinges were therefore forced to form in the beams and all critical sections were well confined. The standard EC8 spectrum defined for medium density soils was used. All infill panels had identical geometry, with dimensions equal to $4200 \times 2750 \times 135$ mm. The thickness resulted from the combination of block thickness (115 mm) plus 10-mm plaster on both sides.

As already indicated, three reinforcement types were then considered: no reinforcement, reinforcement in the mortar layers, and external reinforcement. The mortar layer reinforcement was spaced at 600 mm, resulting in a geometrical percentage between 0.08 and 0.1 %, depending on the

diameter of the bars used (see the previous section). Note that the total steel weight used for each panel was less than 100 N for the case of layer reinforcement.

All tests were performed applying first two vertical loads on the columns, to simulate the presence of the upper storeys. No vertical load was placed on the beam, accepting this small difference from reality. The total vertical load was then kept constant during the tests, allowing the redistribution generated by the application of horizontal forces.

The in-plane tests were performed applying horizontal displacements cycles, according to pre-defined targets between 0.1 and 3.6 % drift, as discussed later. As standard, three cycles were performed at each target displacement.

The out-of-plane tests were performed at different levels of in-plane damage (i.e. after having reached different drift levels in the in-plane tests) .

A summary of the in-plane tests performed is presented in Table 1. The sequence of drift targets has been adjusted during the test campaign, to explore the response for close-to-collapse limit states (drifts at 1.2 % or larger) and for serviceability limit states (drift below 0.4 %). In some cases, the in-plane tests were continued after out of plane expulsion of the infill panels, essentially to verify the ultimate response of the frame alone.

Table 1: Summary of the in-plane tests

Reinforcement	Test n.	Drift						
		0.1%	0.2%	0.3%	0.4%	1.2%	3.6%	0.4%
bare frame	1	3	-	-	3	3	3	3
non-reinforced	2	3	-	-	3	3 *	-	-
φ 6 mm bars	3	3	-	-	3	3	3	-
φ 5 mm bars	4	3	-	-	3 *	-	3	-
mesh	5	3	-	-	3	3 *	3	-
non-reinforced	6	3	3	3	3 *	-	-	-
φ 6 mm bars	7	3	3	3	3 *	-	-	-
φ 5 mm bars	8	3	3	3	3 *	-	-	-
mesh	9	3	3	3	3 *	-	-	-

* = cycles preceding the out-of-plane test

The force-displacement curves obtained from the first five tests are depicted in Figure 2.

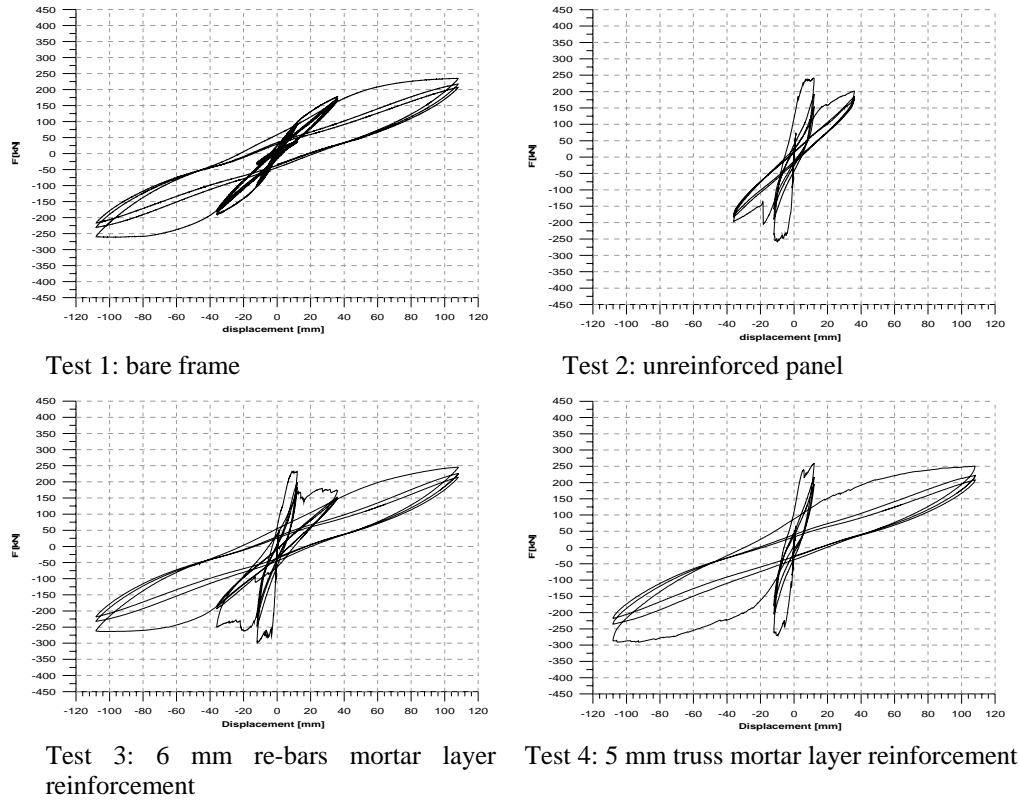


Figure 2: Force-displacement curves obtained from tests

The initial stiffness of infilled frames is significantly larger than the stiffness of the bare frame, with no significant influence from the presence of the reinforcement. In all cases the curves produced by the infilled frame response tend to return to the bare frame curve at drifts values relevant to a collapse limit state, being therefore confirmed that the equivalent period of vibration for high intensity earthquakes is not significantly affected by the presence of the infills, since the effective stiffness essentially corresponds to the stiffness of the bare frame. The energy dissipation for low amplitude cycles is much larger for infilled than for bare frames, confirming that a reduced global response can be expected if local failure modes can be excluded. All kinds of reinforcement are effective in avoiding or significantly reducing the strength deterioration of the panels. The presence of 5 or 6 mm diameter bars, in the form of re-bars or trusses does not significantly affect the results.

The apparent state of damage of the panels at drifts equal to 0.2 % and 0.4 % is depicted in Figure 3. The beneficial effect of a mortar layer reinforcement is evident. The evidence for this latter case is that during the test large cracks open between panel and frame, without significant damage in the panel. Not only then the damage is limited, but also easy to be repaired.

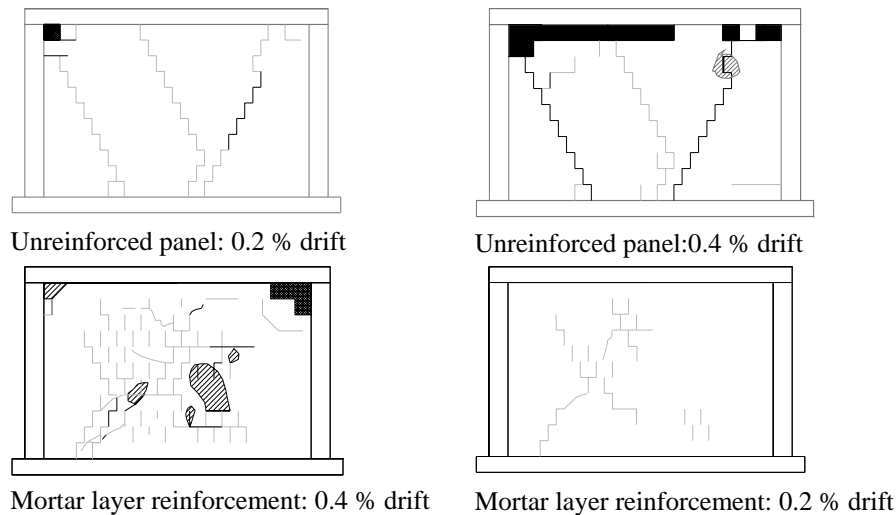


Figure 3: State of damage after three cycles at 0.2 % and 0.4 % drift; test 9 at 2 % drift showed essentially no damage

The main results obtained from the out-of-plane tests are summarised in Table 2 and in Figures 4. The effects of presence of reinforcement and of previous in-plane damage on the failure domain of the panels are evident in the figures and quantitatively shown in the table. For the unreinforced, undamaged panel, the local acceleration required to induce collapse is around 2.5 g, but reduces to 0.5 – 0.7 g after some in-plane damage. The presence of some bed joint reinforcement increases these values approximately to 2.8 and 1.4 g.

The limited displacement capacity of unreinforced panels, results in longer period of vibration, which in turn may result in larger amplification when combined with the main period of vibration of the building. While these figures have obviously a relative meaning, being related to a specific panel geometry, it is immediately clear that very large PGAs would be necessary to induce out-of-plane collapse in reinforced panels.

Table 2: Summary of the out-of-plane tests. For all panels the total area was 11.55 m² and the total mass 1360 kg

Reinforcement	Test n.	Previous in-plane drift	Force [kN]	Displ. ² [mm]	Acceleration ³ [g]	Eq. Pressure ⁴ [kPa]	Stiffness ⁵ [kN/m]	Period [s]
non-reinforced	2	1.2%	6.00	-	0.460	1.00	180	0.546
bars	3	-	-	-	-	-	-	-
Murfor	4	0.4%	17.20	53	1.289	2.87	5160	0.102
mesh	5	1.2%	21.41	22	1.605	3.57	2676	0.142
non-reinforced	6	0.4%	9.00	37	0.675	1.50	720	0.273
bars	7	0.4%	19.70	44	1.477	3.28	2814	0.138
Murfor	8	0.4%	17.50	48	1.312	2.92	1400	0.196
mesh	9	0.4%	46.60	28	3.493	7.77	11184	0.069
non-reinforced ¹	10	-	33.70	13	2.526	5.62	11642	0.068
Murfor ¹	11	-	36.77	45	2.756	6.13	11977	0.067

¹ Test executed on the panel with no damage

² Ultimate displacement

³ Acceleration corresponding to the maximum transversal force (with the total mass participating)

⁴ Equivalent uniform pressure giving to the panel the same maximum bending moment as the four concentrated forces

⁵ Secant stiffness calculated between the 10% and 40% of the strength of the panel

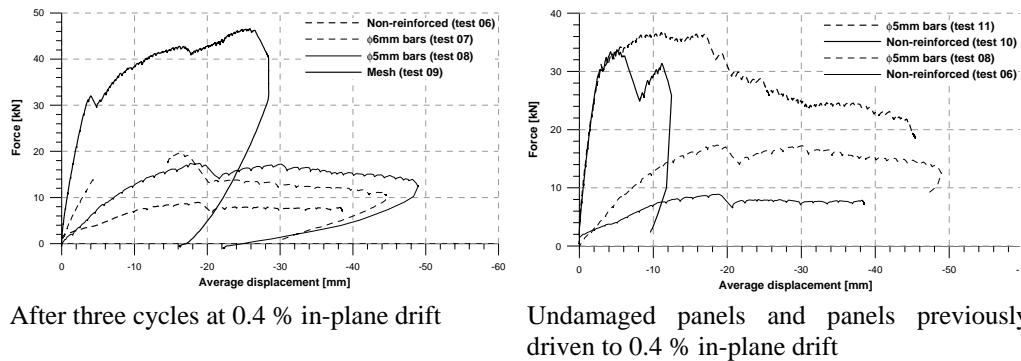


Figure 4: Comparison of force-displacement curves obtained from out-of-plane tests performed on differently reinforced panels

2.2. AAC infill walls with thin mortar layer

The same frame geometry and testing conditions were adopted for the in plane cyclic tests of the infilled frames and for the out-of-plane test on the in-plane damaged infill panels. In this case AAC blocks with thin mortar layer and filled vertical joints were used for all the infill panels with the different reinforcement solutions.

The experimental campaign on AAC masonry infilled r.c. frames included the following reinforcement techniques:

Unreinforced AAC infill panel;

AAC infill panel reinforced by horizontal bars lodged in special grooved blocks every second mortar layer (i.e. with a vertical spacing of 500 mm);

AAC infill panel reinforced by a mid-height r.c. tie beam realized special U-shaped blocks;

AAC infill panel reinforced by flat truss reinforcement every second bed joint;

AAC infill panel with a central opening (door) reinforced by flat truss reinforcement every second bed joint

The AAC blocks used for the tests had the following dimensions: 250 mm height, 300 mm thickness and 625 mm length. U-shaped blocks and blocks with circular vertical holes (150 mm diameter) were used to cast in place the r.c. light frame surrounding the door in the last specimen. No mechanical devices were placed at the interfaces between the frame structure and the infill panel.

The in-plane cyclic force-displacement response of the unreinforced panel is represented in Figure 5 and the final damage is reported in Figure 6.

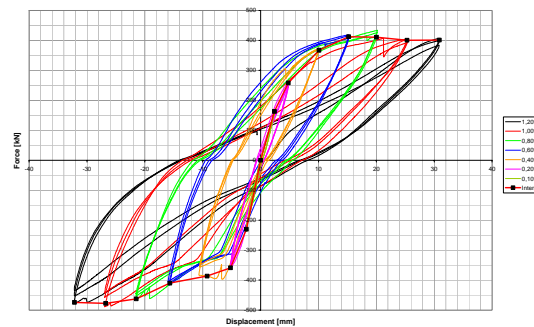
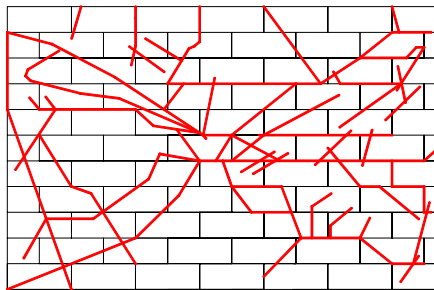


Figure 5. Cyclic force-displacement response of the r.c. frame infilled with the unreinforced AAC masonry panel

With respect to the bare frame, the lateral strength is more than doubled, the initial stiffness significantly increased and the hysteresis cycles enlarged. The test has been carried out up to 1.2 % drift.



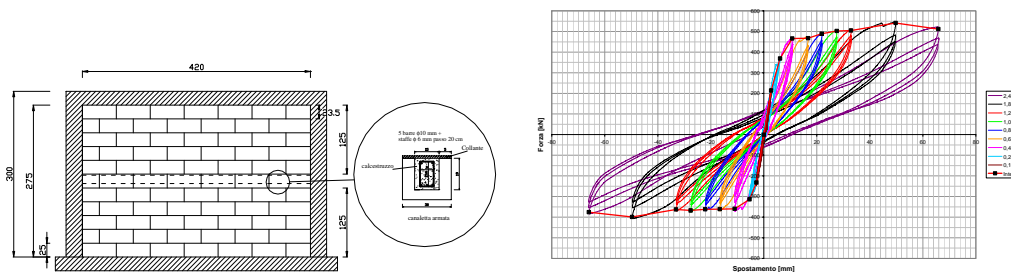
Cracking pattern after 0.8 % drift cycles



The specimen at the end of the in-plane tests

Figure 6. Final damage pattern to the AAC unreinforced infill panel after the in-plane tests

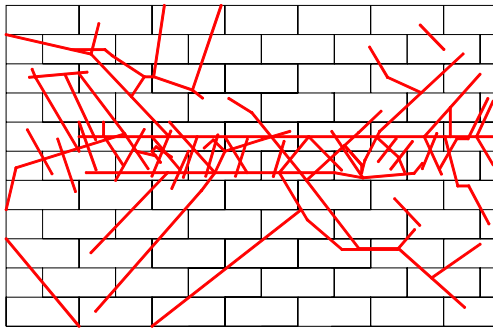
In Figure 7 it is reported the cyclic in plane response of the frame with mid-height reinforcement obtained by r.c. tie beam, cast in place in special U-shaped AAC blocks.



Scheme of the mid-height beam reinforced specimen Cyclic force-displacement curve

Figure 7. Scheme and in-plane lateral response of the mid-height reinforced specimen

The force-displacement response of these tests, carried out up to 3.6 % drift, show a similar cyclic behaviour with a further increase lateral strength. On the other hand, Figure 8 show that a significant damage distribution to the infill panel is concentrated along the r.c. beam and, as shown in Figure 9, cracks also occur in the frame columns



Cracking pattern after 1.0 % drift cycles



The specimen at the end of the in plane tests

Figure 8. Final damage pattern to the AAC infill panel with mid-height tie beam after the in-plane tests



Figure 9. Damage to the frame columns due to the mid-height r.c. beam

In Figure 10 it is reported the cyclic in plane response of the frame with bed joint reinforced infill panel by means of grooved special blocks every second layer.

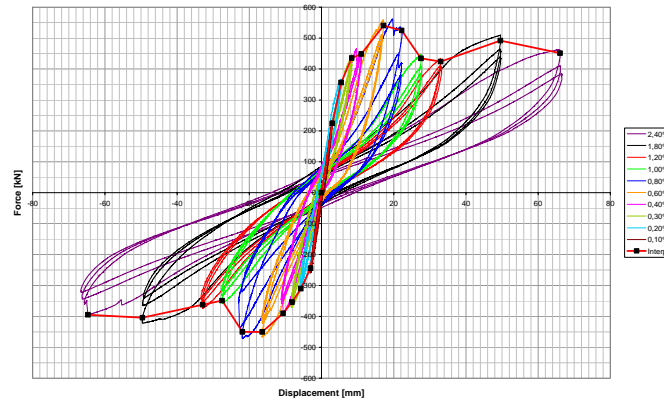
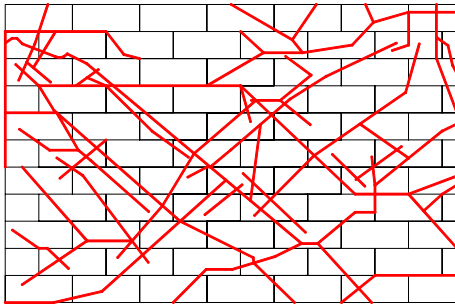


Figure 10. Cyclic force-displacement response of the bed joint reinforced infilled frame (rebars in special blocks)

The initial stiffness is not affected by the presence of the reinforcement, the displacement-ductility capacity is increased and a softening branch of the envelope curve can be related to the observed shear failure mode (Figure 11).



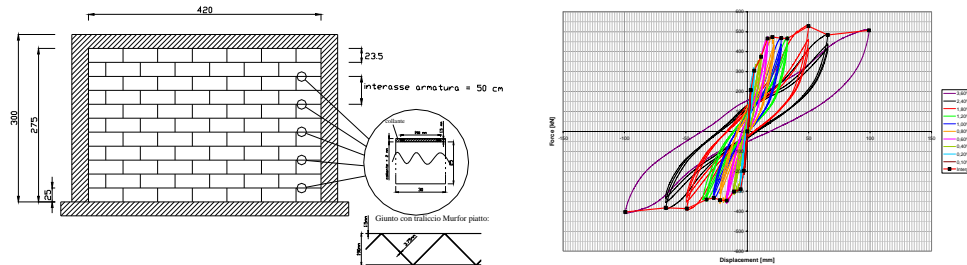
Cracking pattern after 1.8 % drift cycles



The specimen at the end of the in plane tests

Figure 11. Final damage pattern to the AAC infill panel with rebar reinforcement

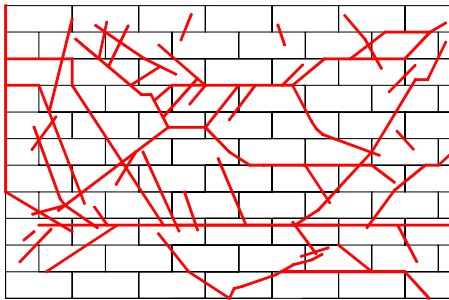
In Figure 12 it is reported the cyclic in plane response of the frame with horizontal flat truss reinforcement placed every second mortar layer.



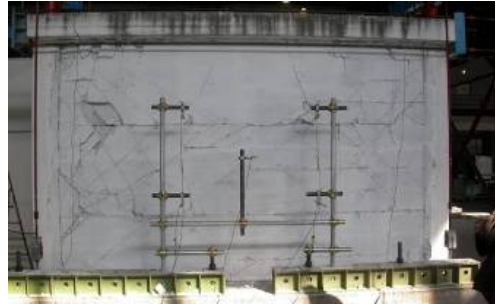
Scheme of the flat truss horizontally reinforced specimen Cyclic force-displacement curve

Figure 12. Scheme and in-plane lateral response of the AAC infilled frame with flat truss reinforcement

The initial stiffness is not affected by the presence of the reinforcement, the displacement-ductility capacity is increased and the strength plateau is due to the sliding mechanism, with friction, on the unreinforced mortar joints.



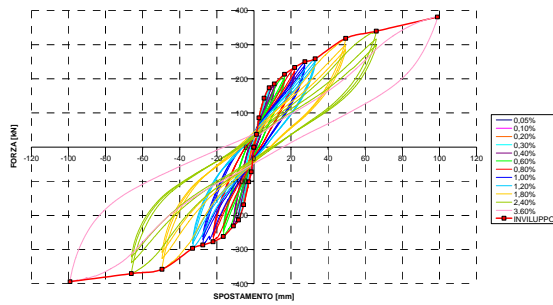
Cracking pattern after 1.0 % drift cycles



The specimen at the end of the in plane tests

Figure 12. Final damage pattern to the AAC infill panel with flat truss reinforcement

Figure 13 shows the cyclic the final damage pattern and the force-displacement curve obtained for the horizontally reinforced AAC infilled frame with a central door. Although the lateral strength is obviously reduced, it is anyhow significant and the displacement capacity is not affected by the presence of the opening. The presence of the bed joint reinforcement controls the damage to the infill panel.



Cyclic force-displacement curve



The specimen at the end of the tests

Figure 13. Results of the test on the infilled frame with a central opening and the horizontal flat truss reinforcement

A comparison of the envelope curves of the whole experimental campaign is presented in Figure 14.

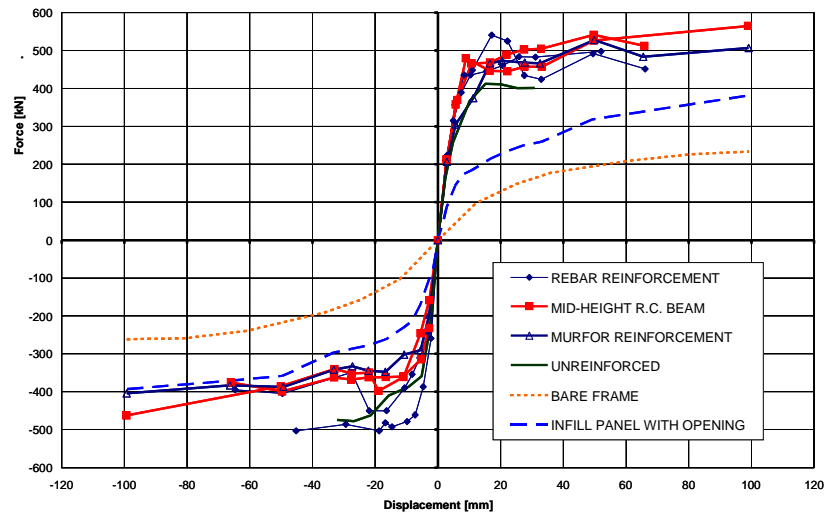


Figure 14. Force-displacement envelope curves of the performed tests

3. DESIGN OF INFILL BED JOINT REINFORCEMENT

The design of bed joint reinforcement for infill panels is usually governed by the need of reducing their out-of-plane vulnerability. For these reason the application reported in the following refer to design tools and applications aiming at this scope.

3.1. Seismic design tool for steel truss reinforcement

A simple Excel-based tool for seismic design of infill bed joint reinforcement has been developed by EUCENTRE for Bekaert, according to the EC8 and Italian Seismic Code Ordinanza 3274.

JOB TITLE	Title	
OWNER	Owner	
DESIGNER	Designer	

STRUCTURE		
Structural typology		
FRAME		
Type of FRAME structure		
RC FRAME		
Height of the structure from the rigid basement [m]		
12	H	
Not necessary		
10	At	
Not necessary		
10	nw	
Not necessary		
8	lw	

INFILL WALLS		
Characteristic strength in compression of the wall [MPa]		
1.47		
Weight of the wall for unit of volume [kN/m ³]		
4.85		
Height of the units [m]		
0.15		
Thickness of the wall [m]		
0.12		
Length of the wall [m]		
4.15		
Height of the wall [m]		
2.75		

MURFOR		
Type		
RND/Z RND/E RND/S		
Diameter [mm]		
5		
Width [mm]		
180		

SEISMIC INPUT		
Magnitude of the earthquake		
> 5.5 Richter		
Peak ground acceleration / g		
0.25		
Soil type according to EC8 provisions		
C		
Importance factor of the element		
All the other cases		

RESULTS		
Typological dynamic coefficient	Ct = 0.075	
Structural period of vibration	T1 = 0.484 sec	
Typological dynamic coefficient		
Structural period of vibration		
Masonry elastic modulus	E = 1323 MPa	
Masonry density	m = 0.494 tons/m ³	
Infill period of vibration	Ta = 0.1935 sec	
Soil amplification factor	S = 1.15	
Importance factor	γ _s = 1	
Structural amplification factor	Sa = 1.125	
Out-of-plane seismic force	F = 7.47 kN	
Maximum vertical spacing required between the Murfor elements	S _{max} = 500 mm i.e. every 3 layers of	

Figure 15. Layout of the infill reinforcement design tool

The structural period of vibration is evaluated according to the EC8 simplified formula

$$T_1 = C_t \cdot H^{\frac{3}{4}}$$

where C_t is a coefficient depending on the structural typology and construction material, while H is the total height of the building.

The modulus of elasticity is evaluated as a function of the compressive strength.

In order to evaluate the dynamic interaction effects between the building structure and the infill wall, it is necessary to evaluate the out-of-plane period of vibration of the infill panel. According to the simplified hypothesis of a panel restrained to the frame structure by cylindrical hinges, the period of vibration can be approximated by the following formula:

$$T_p = \frac{2}{\pi} \cdot \left(\frac{1}{L^2} + \frac{1}{H^2} \right) \cdot \sqrt{\frac{E \cdot t^3}{12 \cdot m}}$$

Where L , H and t are panel length, height and thickness respectively and E is the masonry elastic modulus, m the mass of a volume unit.

The reinforcement trusses can be subdivided into two typologies depending on the type of line wirecross section. Horizontal mortar layers can be indeed traditional (filled with general purpose mortar) or very thin ones (with glue-mortar).

The considered seismic action is characterised by an expected maximum Magnitude and by a soil class.

The importance factor of the non-structural elements is calculated in accordance with the typology as defined in EC8 (4.3.5.3).

Consistently with the previous assumptions it is possible to define the following values:

Non-structural element importance factor

Peak Ground Acceleration

Soil amplification coefficient

The adopted panel behaviour factor ($q = 2$)

The equivalent seismic force acting on the infill wall orthogonally to its plane is given by the following formula, keeping into account the dynamic interaction between global structural response and infill panel behaviour:

$$F_a = W_a \cdot S_a \cdot \gamma_I / q_a$$

where W_a is the element weight, g_I and q_a are the importance factor and the behaviour factor respectively.

The amplification coefficient S_a is given by:

$$S_a = \frac{a_g \cdot S}{g} \cdot \left(\frac{3 \cdot (1 + Z/H)}{1 + (1 - T_a/T_1)^2} - 0.5 \right) \geq \frac{a_g \cdot S}{g}$$

where $a_g \cdot S / g$ is the ratio between the maximum structural acceleration and gravity, Z/H is the ratio between the panel centroid height from the foundation and the total height of the building (assumed

equal to 1 for sake of safety), T_a and T_1 are the panel vibration period and the structural vibration period respectively.

The ultimate resisting moment is evaluated according to the hypothesis of 'horizontal beam': by this calculations the maximum resulting vertical spacing of Murfor joint reinforcements is obtained: a minimum vertical spacing of 500 mm is considered according to what reported in the Italian seismic code, Annex 2 of OPCM 3274/03, at point 5.6.4.

The described method, for safety sake, takes into account the effect of a previous in-plane damage corresponding approximately to a 0.4% in-plane drift.

4. MASONRY PIERS WITH BED JOINT REINFORCEMENT

Reinforced masonry solutions typically provide that horizontal reinforcement solution are combined with vertical reinforcement (Tomazevic, 1999 [12]).

Vertical reinforcement improve the flexural lateral strength of a masonry pier, with a significant upgrading of the cyclic response.

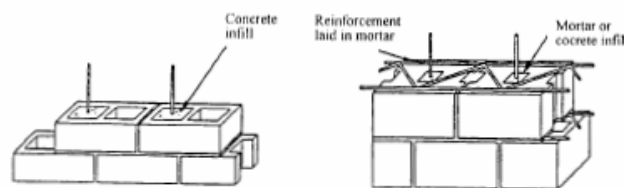


Figure 16. Reinforced masonry (after [12])

The use of bed joint reinforcement is certainly the easiest possibility to provide horizontal reinforcement. It provides cracking control, flexural reinforcement in masonry spandrel beams and lintels and, above all, shear reinforcement in masonry piers.

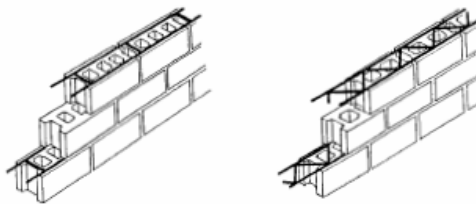


Figure 17. Truss type bed joint reinforcement (after [12])

The truss type reinforcement solution (Figure 17) can be a valid option both for its superior structural effectiveness and in case of unconnected multi-layered walls (Figure 18).

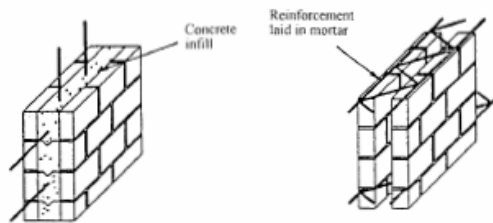


Figure 18. Use of truss type reinforcement in multi-layered walls (after [12]).

Vertical reinforcement require a different constructive sequence, proper detailing of anchorages and overlapping zones and the use of special blocks with vertical holes which are typically filled with concrete.

Bed joint reinforcements, on the other hand, do not require masonry execution modifications, nor special anchoring solutions. For these reasons the use of this kind of reinforcement is typically well accepted, also by the workmanships.

Therefore, because of the stated advantages, structural reinforced masonry schemes with only bed joint reinforcement would be very interesting.

The contribution of horizontal reinforcement to the shear resistance of masonry piers does not really requires the presence of vertical rebars.

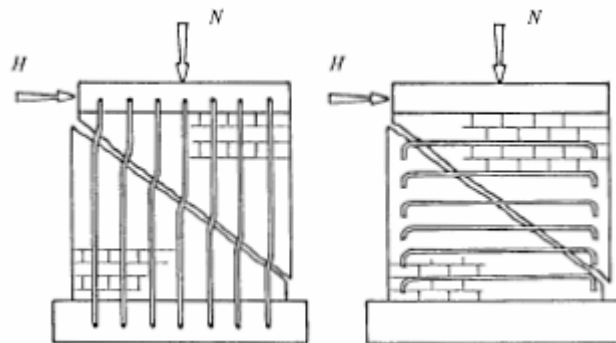


Figure 19. Influence of diffused vertical and horizontal reinforcement on the shear failure mode (after [10])

The experimental response of horizontally reinforced masonry was initially investigated by Tomazevic and Zarnic [13]. Penna et al. [9] have recently compared, through an extensive experimental campaign on full scale AAC bearing walls, the role played by horizontal reinforcement in improving the seismic performances of such masonry typology.

4.1. Experimental research on bed joint reinforced AAC masonry piers

The experimental campaign performed at EUCENTRE included in plane cyclic tests on full scale masonry piers with the following structural solutions:

Unreinforced masonry;

Confined masonry with horizontal truss-type bed joint reinforcement;

Horizontally reinforced masonry, with flat truss steel reinforcement or rebar reinforcement placed in grooved blocks.

The dimensions of the blocks is the same used for the infill panel tests.

Three in-plane slenderness configurations have been considered and two levels of vertical force have been applied and kept constant during the test.

A total of 9 specimen have been tested, with constant height (2.75 m) and thickness (30 cm), and lengths ranging from 1.5 m to 4.5 m.

By means of hydraulic jacks the static vertical loading of 200 kN and 300 kN has been applied. The considered experimental configurations are reported in the matrix in Figure 20.

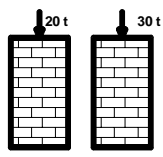
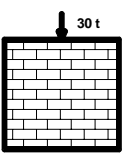
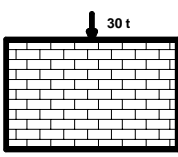
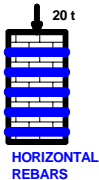
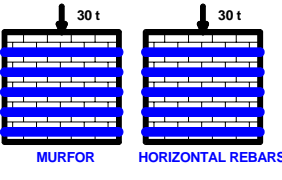
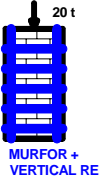

	1500 mm x 2750 mm	3000 mm x 2750 mm	4500 mm x 2750 mm
UNREINFORCED MASONRY			
PARTIAL REINFORCEMENT	 HORIZONTAL REBARS	 MURFOR HORIZONTAL REBARS	
REINFORCED MASONRY	 MURFOR + VERTICAL REBARS	 MURFOR + VERTICAL REBARS	

Figure 20. Matrix of the considered experimental schemes

In Figures 21 and 22 a synthetic comparison of the experimental results, in terms of lateral force-displacement envelope curves in presented, squat walls (3 m x 2.75 m) and slender walls (1.5 m x 2.75 m), respectively.

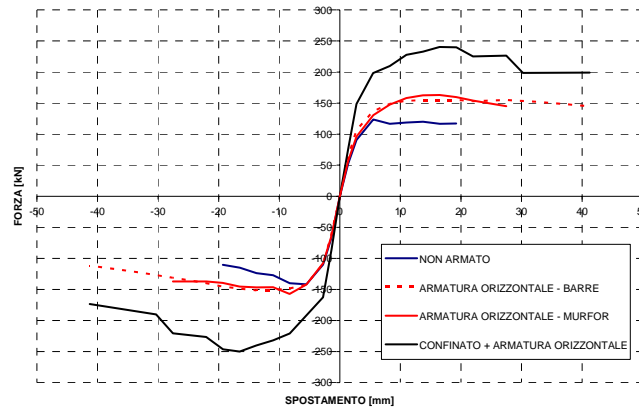


Figure 21. Force-displacement envelopes for the “squat” AAC walls (blue: unreinforced; black; confined and horizontally reinforced; red: horizontally reinforced).

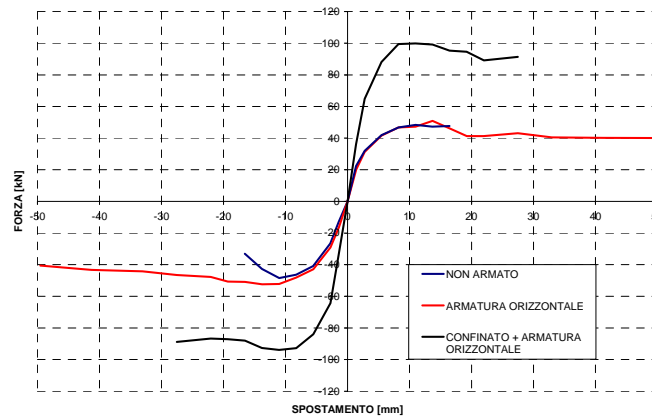


Figure 22. Force-displacement envelopes for the “slender” AAC walls (blue: unreinforced; black; confined and horizontally reinforced; red: horizontally reinforced).

All the slender walls have shown a clear flexural behaviour. The confined masonry solution provided, with respect to the unreinforced one a significant increment of strength and stiffness and a relevant upgrading of the displacement capacity. In the case of the bed joint reinforced wall, the strength and stiffness has been not altered but, due to the reduction of vertical and diagonal cracking, the lateral displacement capacity has been highly increased.

The unreinforced squat wall has shown a clear shear failure, the horizontally reinforced one a flexural one and for the confined wall a mixed failure mode has been observed. The confined masonry solution provided, with respect to the unreinforced one, a significant increment of strength and displacement capacity. In the case of the bed joint reinforced wall, the stiffness has been not modified but, due to the prevention of the shear failure mode, the lateral strength has been slightly increased and the displacement capacity has been enlarged to the same level of the confined solution.

4.2. Proposed criterion for shear strength of horizontally reinforced masonry piers

In order to assess the lateral strength of masonry piers with only bed joint reinforcement, the following shear strength criterion is proposed:

$$V_R = V_{R,M} + V_{R,H} = f_v l' t + f_y A_{sh} d' / s \leq f_v l t$$

Where f_v is the masonry shear strength,
 l is the wall length,
 l' is the length of the compressed portion of the wall,
 f_y is the steel yield strength,
 A_{sh} is the area of the cross section of the bed joint reinforcement,
 s is the vertical spacing of the bed joint reinforcement,
 h is the wall height
 and $d' = \min(l', h)$

The results of such a criterion, combined with the flexural strength criteria, well match the experimental values obtained for the AAC masonry walls, but further investigations and tests should be addressed to this direction.

5. CONCLUSIONS

Bed joint reinforcement is a easy and practical technique which provides significant improvements to the in plane and out of plane performances of both masonry walls and infill panels.

In the infill panels with bed joint reinforcement there is an evident increase in strength, ductility, energy dissipation capacity and the beneficial effects on the frame structural response also include a more regular response and damage limitations both to the panel and to the frame. A easy to use excel based tool has been presented in the paper. It allows the designer to calculate the required bed joint reinforcement in order to prevent damage and collapse to the infill panels.

Horizontally reinforced masonry (HRM) can be a valid alternative to currently used unreinforced, confined and vertically reinforced masonry. The obtained experimental results show, in particular, the great increment in displacement capacity and the significant contribution to the shear strength. Truss-type reinforcements appear equivalent to rebars, but they are easier to use, more regular and requiring shorter anchorage lengths are more effective in controlling cracking.

A specific strength criterion has been developed and presented in the paper, in order to provide simple calculation rules which can be well combined to the easiness of construction which can promote this technology in different countries in the world, improving structural and seismic safety.

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