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MASONRY SHEAR WALLS SUBJECTED TO CYCLIC LOADING: INFLUENCE OF CONFINEMENT AND HORIZONTAL REINFORCEMENT

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Abstract

The behavior of masonry shear walls is fundamental in the design of masonry buildings subjected to different actions, namely of seismic nature. The usage of unreinforced, confined or reinforced masonry is currently subjected to a strong debate in Europe due to the new codes. In particular, the part of Eurocode 8 (Design of structures for earthquake resistance) related to masonry structures is only a limited compromise for the different countries. A large testing program was started at University of Minho in order to clarify issues regarding confined masonry and unfilled vertical joints. Confined masonry is assumed as a hybrid material joining masonry with small section horizontal and vertical lightly reinforced concrete elements. This project, partly sponsored by the light-weight concrete block industry, aims at defining adequate structural solutions for regions of low to high seismicity in Portugal.

This paper discusses the results of the experimental program, consisting mainly of masonry walls subjected to cyclic actions and constant pre-compression. Sixteen specimens are considered, being the shear strength, ductility, energy dissipation and stiffness discussed. The key aspects under discussion are: (a) the possibility of replacing the filling of the vertical joints by interlocking and horizontal bed joint reinforcement, (b) the need for filling vertical joints in confined masonry solutions.

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Introduction

In the last decades, the study of masonry walls subjected to combined in plane normal and shear loading received much attention. A few examples of different experimental and numerical research includes:
1973, Meli carried out tests in confined masonry, assessing the shear strength, ductility and energy absorption.
1974, Williams and Scrivener studied the influence of repeated loading and ductility.
1980, Bernardini et al. addressed the stiffness degradation, crack evolution and energy dissipation.
1986, Lüders and Hidalgo performed cyclic tests in partly and fully grouted walls, addressing the effect of reinforced horizontal joints.
1988, Tomaževič and Lutman studied the seismic resistance of reinforced masonry walls.
1992, Sanchez et al., and Astroza et al., studied the behavior of confined masonry under cyclic loading and quantified the confining improvement.
1990, Paulson and Abrams assessed the static and dynamic response of model buildings.
1990, Shing et al. addressed the influence of the loading history and the percentage of horizontal reinforcement in the shear strength.
1996, Tomaževič et al. discussed the influence of horizontal load application procedure in the shear strength, failure mode and energy dissipation capacity.
2004, Bosiljkov assessed the influence of the type of joints in the shear strength of walls.

The new European codes of design (EC6 and EC8) should allow innovation and optimization of masonry construction technology and masonry products that do not deviate significantly from current building practice. Portugal did not have a national masonry code due, as most modern building structures are in steel or concrete. Different economic studies indicate the feasibility of unreinforced and confined masonry structures for low and medium rise buildings. Consequently, there is a lack of experience and knowledge in using structural masonry. For this reason, the light-weight block industry decided to co-sponsor a industrial project aiming at defining adequate structural masonry solutions, taking into account normal building practices, seismic hazard and the industrial technological possibilities. This paper presents the preliminary results of tests on sixteen shear walls of various design configurations.

Description of experiments

Testing Program

The testing program included 16 walls, scaled 1:2, as shown in Table 1. The walls have been designed before testing so that a shear failure mode would be obtained instead of a flexural or rocking mode. Two unreinforced walls configurations have been considered, assuming filled and unfilled vertical joint. In the latter, the benefit of using bed joint reinforcement was analyzed. Such configurations have been tested again using confined masonry, always assuming unfilled vertical joints. In wall W6, the horizontal bed joint reinforcement is properly fixed to the reinforced concrete confining elements. Figure 1 presents the different type of masonry specimens.

Type and designation of wall			no.	Block	Mortar	Horizontal reinforcement			Confining elements
unreinforced	standard unreinforced	W2.1	4	B1	✓				
unreinforced	standard unreinforced	W2.2	3	B1	✓	✓			
unreinforced	lightly horizontally reinforced	W2.3	2	B1	✓		✓		
Confined	lightly horizontally reinforced	W2.5	3	B1	✓		✓		✓
Confined	lightly horizontally reinforced	W2.6	2	B1	✓		✓	anchor	✓
Confined	standard confined	W2.4	2	B1	✓				✓

Table 1. Type and designation of specimens.

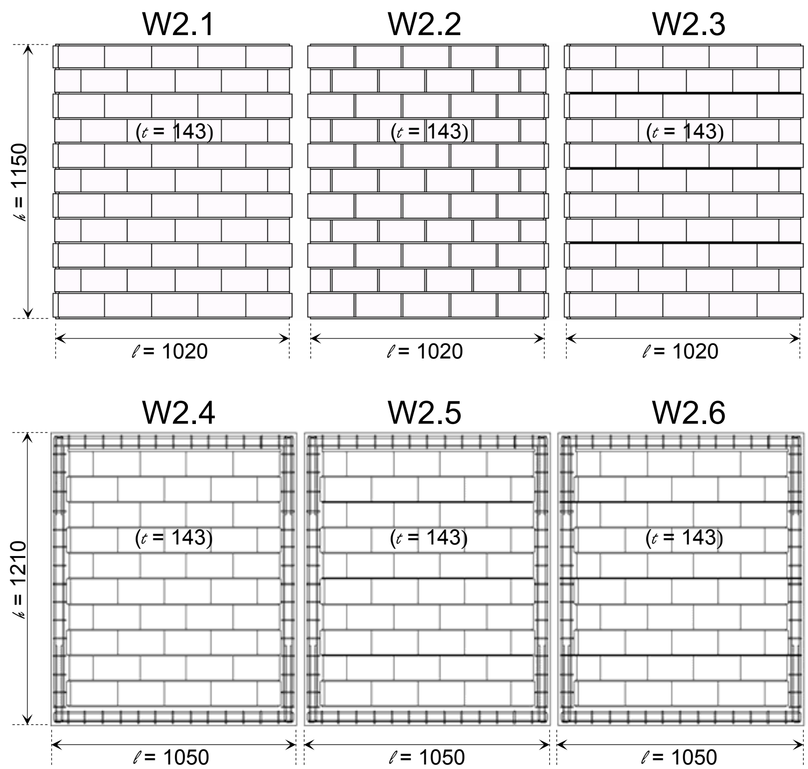

Figure 1. Type of masonry specimens.

Masonry Materials

The blocks adopted in the testing program are regularly produced by the industry to comply with thermal regulations and has nominal dimensions of $200\times320\times100mm^3$. This is a half block in terms of height and length for regular blocks ($400\times320\times200mm^3$). After cutting this half block in two pieces, the resulting half scale block used in the walls has dimensions of $200\times143\times100mm^3$, as shown in Figure 2. The normalized compressive strength of the block is $5.7N/mm^2$. The mortar adopted for the wall construction was a pre-mixed mortar, type MAXIT AM10 and produced by MAXIT Group, with a 10 N/mm^2 compressive strength.

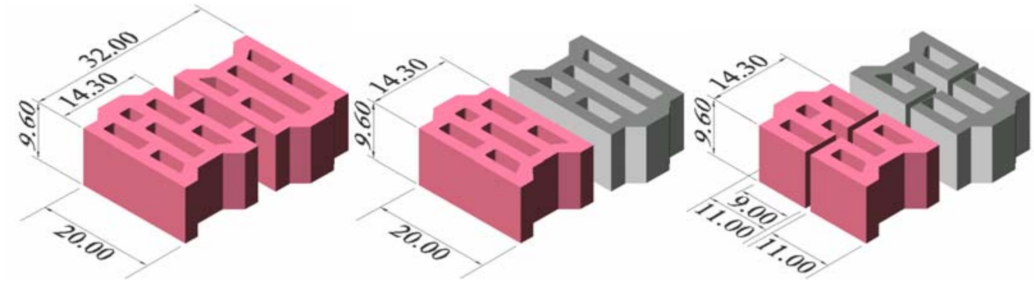

Figure 2. Standard block, Block and Half-block used in the tests.

Confining concrete elements have been made using self compacting concrete (f_c = 31.5N/mm^2) with a transverse section of 143×75mm^2 (vertical elements) and 143×80mm^2 (horizontal elements). The reinforcement consists of 4ϕ6mm bars (f_{yk}=400N/mm^2, ρ_L=0.151%) with ϕ4mm stirrups spaced at 75mm (f_{yk}=400N/mm^2, ρ_V=0.29%).

Bed joint reinforcement is of truss type, Murfor® produced by Bekaert, including two longitudinal bars of 5 mm (f_{yk}=550N/mm^2, ρ=0.09%).

Test set-up and cyclic loading

The walls were tested using the lay-out shown in Figure 3, where the lower steel beam was fixed to the strong floor in order to preclude any movement. The vertical load was applied with an actuator of 350kN, designed to keep the vertical load constant. Therefore, vertical displacements are allowed in the top steel beam. It is noted that rotation of the top beam is not fully prevented and the values of vertical displacements at the edges of the top beam have been recorded. Steel rollers were placed between the top steel beam and a load distributor beam to reduce the friction induced by lateral wall movement. The horizontal cyclic load was applied using a 250 kN actuator fixed to the strong wall at mid-height of the specimen.
The tests were carried out with constant vertical stress, at a constant level of about 30% of the masonry strength, 0,3 × 3,2 = 0,9 N/mm^2. The horizontal action is applied to the wall via controlled displacement at a rate of 60μmm/s. Two full displacement cycles were programmed for each amplitude increment, aiming at strength and degradation assessment (Calvi 1996; Tomaževic 1996; Vasconcelos 2005).

Test results

Key Parameters

The behavior of masonry walls is characterized by key parameters, typically, maximum shear resistance, horizontal displacements at selected load levels, ductility and energy dissipation (Bosiljkov 1988, Magenes 1992). In order to obtain such reference values the envelope of the response of each wall was determined. This envelope is made from the value of the shear load H and the horizontal displacements d. Characteristic points of the diagram, include the occurrence of the first crack d_{cr}, the maximum shear load H_{max} and the corresponding lateral displacement d_{max}.

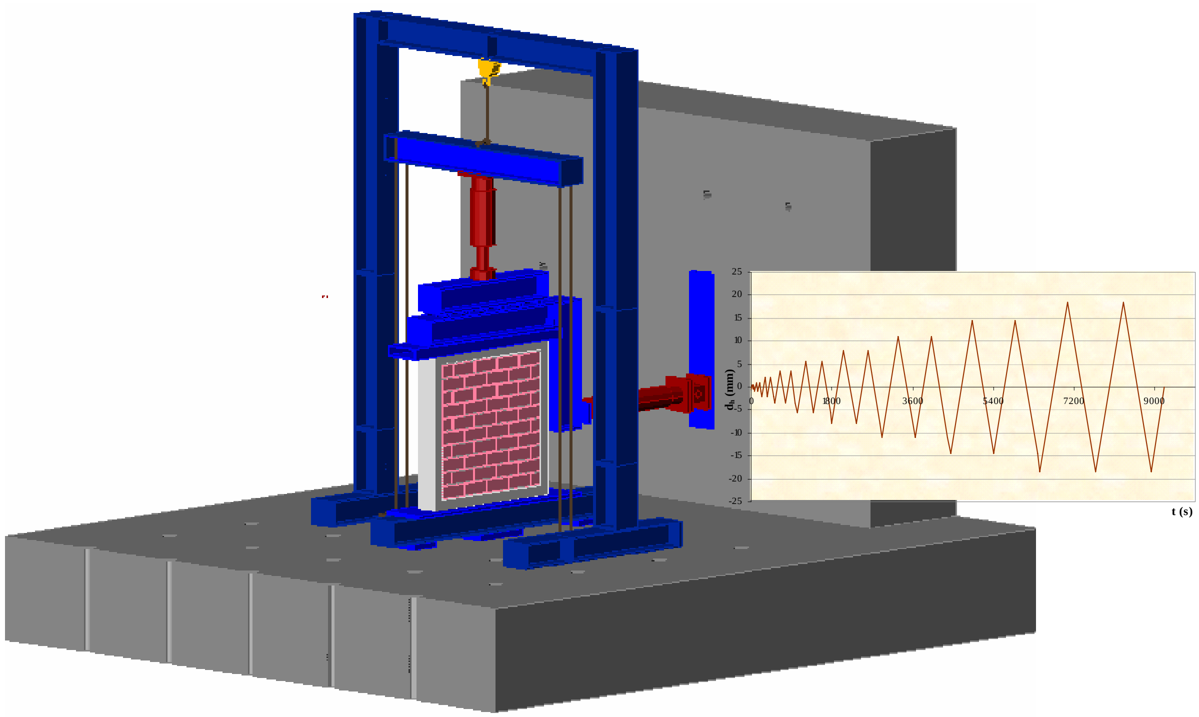

Figure 3. Test set-up and Typical displacement time history.

The stiffness of the wall is defined as the slope of the diagram $H - d$, given by

$$K = \frac{H}{d} \qquad \textbf{[1]}$$

The elastic wall stiffness (K_e) is defined using the early load values, where the response is linear, whereas the stiffness K_{Hmax} is the secant stiffness corresponding to the occurrence of the maximum lateral load.

The deformation capacity is assessed in terms of horizontal displacement achieved and ductility. Here, ductility is defined as the relation between the maximum theoretical displacement d_u and the linear elastic displacement d_e. These values are obtained from the bi-linear diagram shown in Figure 4, where the area under the bi-linear diagram is calculated so that the energy dissipated experimentally is replicated.

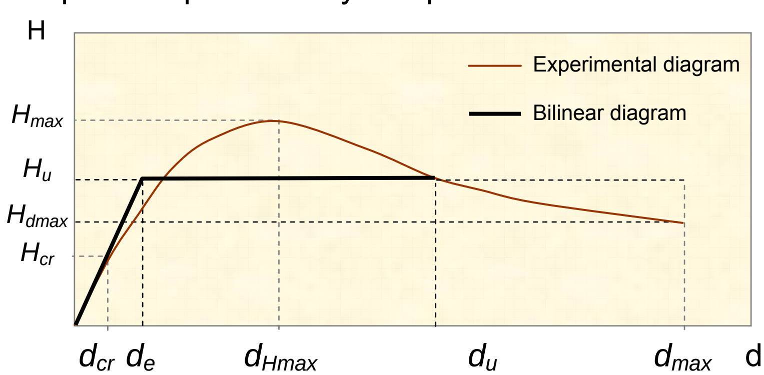

Figure 4. Envelope of experimental values and bilinear diagram.

With the average value from each wall series, it was possible to obtain the pair of values leading to crack initiation (H_{cr}, d_{cr}), maximum horizontal force (H_{max}, d_{Hmax}) and maximum displacement (d_{max}), as well as the maximum theoretical displacement d_u. These values allow to calculate stiffness $K_{e,}$ stiffness K_{Hmax}, maximum theoretical force H_u and ductility μ, given by:

$$K_e = \frac{H_{cr}}{d_{cr}}\ ;\ A_{real}^{envelope} = \frac{H_u \times d_e}{2} + H_u \times (d_{\max} - d_e)\ ;\ K_e = \frac{H_u}{d_e}\ ;\ \mu = \frac{d_u}{d_e} \qquad \textbf{[2]}$$

It is usual to further normalize the horizontal displacement by the wall height, given by

$$drift_{1s\ crack} = \frac{d_{cr}}{h}\ ;\ drift_{H_{max}} = \frac{d_{H\max}}{h} \qquad \textbf{[3]}$$

for the maximum horizontal force.

The different values obtained in the current testing program are given in Table 2, grouped by wall type.

Table 2. Parameters of lateral resistance and deformability.

Wall group	H_{cr} (kN)	d_{cr} (mm)	K_e (kN/mm)	H_{max} (kN)	d_{Hmax} (mm)	K_{Hmax} (kN/mm)	d_{max} (mm)	d_e (mm)	H_u (kN)	d_u (mm)	μ	A_{real} (kN.mm)	*lateral drift* $1_{st\ crack}$	H_{max}
W2.1	50.41	0.72	70.43	**80.94**	2.28	35.58	8.08	0.86	60.49	4.83	**5.62**	462.50	0.06%	**0.20%**
W2.2	63.78	0.82	77.42	**88.85**	2.63	33.73	9.28	0.82	63.31	5.26	**6.43**	561.72	0.07%	**0.23%**
W2.3	46.02	0.86	53.60	**93.11**	4.71	19.78	11.79	1.32	70.75	7.26	**5.50**	787.63	0.07%	**0.41%**
W2.4	57.68	0.79	72.58	**95.02**	3.09	30.74	12.80	0.95	69.12	6.93	**7.27**	851.67	0.07%	**0.26%**
W2.5	54.80	1.04	52.53	**113.73**	5.43	20.93	15.62	1.51	79.53	9.68	**6.39**	1181.91	0.09%	**0.45%**
W2.6	62.14	1.06	58.64	**121.75**	5.84	20.86	16.32	1.61	94.54	10.12	**6.28**	1467.09	0.09%	**0.48%**

Failure Mechanisms

Figure 5 illustrates the typical failure modes obtained for the walls tested. In the walls without bed joint reinforcement (W2.1, W2.2 and W2.4), initially flexural behavior dominates with horizontal cracks appearing at the bottom and top of the walls. With increasing application of horizontal displacement, a diagonal shear crack appears, usually well defined and with sudden occurrence for a given orientation of the loading. With the load increase and inversion of load direction, additional diagonal cracks appear.

In the walls with light bed joint reinforcement, the strength deterioration is slow and more distributed cracking occurs (Zepeda 2000). At ultimate stage, cracking is much more severe as the ultimate displacement is much larger. In walls W2.5 and W2.6, the steel bars of the confining elements are severely stressed, with considerable cracking of these elements. In these walls, masonry crushing was also observed at final stage due to the larger number of cycles applied. In the unreinforced masonry wall (only with light bed joint reinforcement), a higher maximum load could be reached but the test was terminated due to danger of damaging the test set-up with an uncontrolled failure.

Figure 5. Typical failure mechanisms.

Lateral Resistance and Deformability

Table 3 presents a comparative analysis of the results, including filled vs. unfilled vertical joints, confined vs. unreinforced masonry walls, and the effect of including bed joint reinforcement. Figure 6 presents the envelopes of the experimental load-displacement

diagrams and the bilinear diagrams, for all the walls. From these results, the following observations can be made:
- The addition of bed joint reinforcement in standard unreinforced masonry contributes to a very low increase of the shear resistance (5 to 10%). The horizontal cracking displacements are also increased marginally d_{cr}, with a typical drift at peak of 0.21%. On the contrary, the peak displacement d_{Hmax} and maximum theoretical displacement d_u are significantly increased (20% to 100%)
- The addition of bed joint reinforcement in confined masonry contributes to a moderate increase of the shear resistance (about 20%).Confined masonry walls have a shear strength increase of about 20%, when compared to unreinforced masonry. The horizontal displacements increase also, leading to a ductility about 20% larger than unreinforced walls. The typical drift at peak is about 0.45%. Again, the peak displacement d_{Hmax} and maximum theoretical displacement d_u are significantly increased (30% to 80%)
- The theoretical resistance (using the bilinear diagram) is about 75% of the maximum experimental resistance.

Table 3. Comparison lateral resistance and deformability.

Comparison between wall groups	H_{cr} (–)	H_{max} (–)	H_u (–)	$H_{average}$	d_{cr} (–)	d_{Hmax} (–)	d_u (–)	$d_{average}$	μ	lateral drift $1_{st\ crack}$	lateral drift H_{max}
filled / unfilled vertical joints											
W2.2/W2.1	1.27	**1.10**	1.05	1.14	1.15	**1.16**	1.09	1.13	**1.14**	1.15	**1.16**
confined wall / unreinforced wall											
W2.4/W2.1	1.14	**1.17**	1.14	1.15	1.11	**1.36**	1.43	1.30	**1.29**	1.06	**1.29**
W2.5/W2.3	1.19	**1.22**	1.12	1.18	1.22	**1.15**	1.33	1.23	**1.16**	1.15	**1.10**
average	1.17	**1.20**	1.13	1.17	1.16	**1.26**	1.38	1.27	**1.23**	1.11	**1.19**
effect of bed joint reinforcement											
W2.3/W2.1	0.91	**1.15**	1.17	1.08	1.20	**2.07**	1.50	1.59	**0.98**	1.20	**2.07**
W2.5/W2.4	0.95	**1.20**	1.15	1.10	1.31	**1.76**	1.40	1.49	**0.88**	1.31	**1.76**
W2.6/W2.4	1.08	**1.28**	1.37	1.24	1.33	**1.89**	1.46	1.56	**0.86**	1.33	**1.89**
average	0.98	**1.21**	1.23	1.14	1.28	**1.90**	1.45	1.55	**0.91**	1.28	**1.90**

Figure 7 presents the bilinear diagrams for all wall groups, which further confirm a better qualitative behavior of the walls, according to the following rank (from less to most favorable response): 1. Unreinforced masonry with unfilled vertical joints; 2. Unreinforced masonry, with filled vertical joints; 3. Confined masonry with unfilled vertical joints; 4. Unreinforced masonry with light bed joint reinforcement and unfilled vertical joints; 5. Confined masonry with light bed joint reinforcement and unfilled vertical joints; 6. Confined masonry with light bed joint reinforcement, anchored to the confining elements, and unfilled vertical joints.

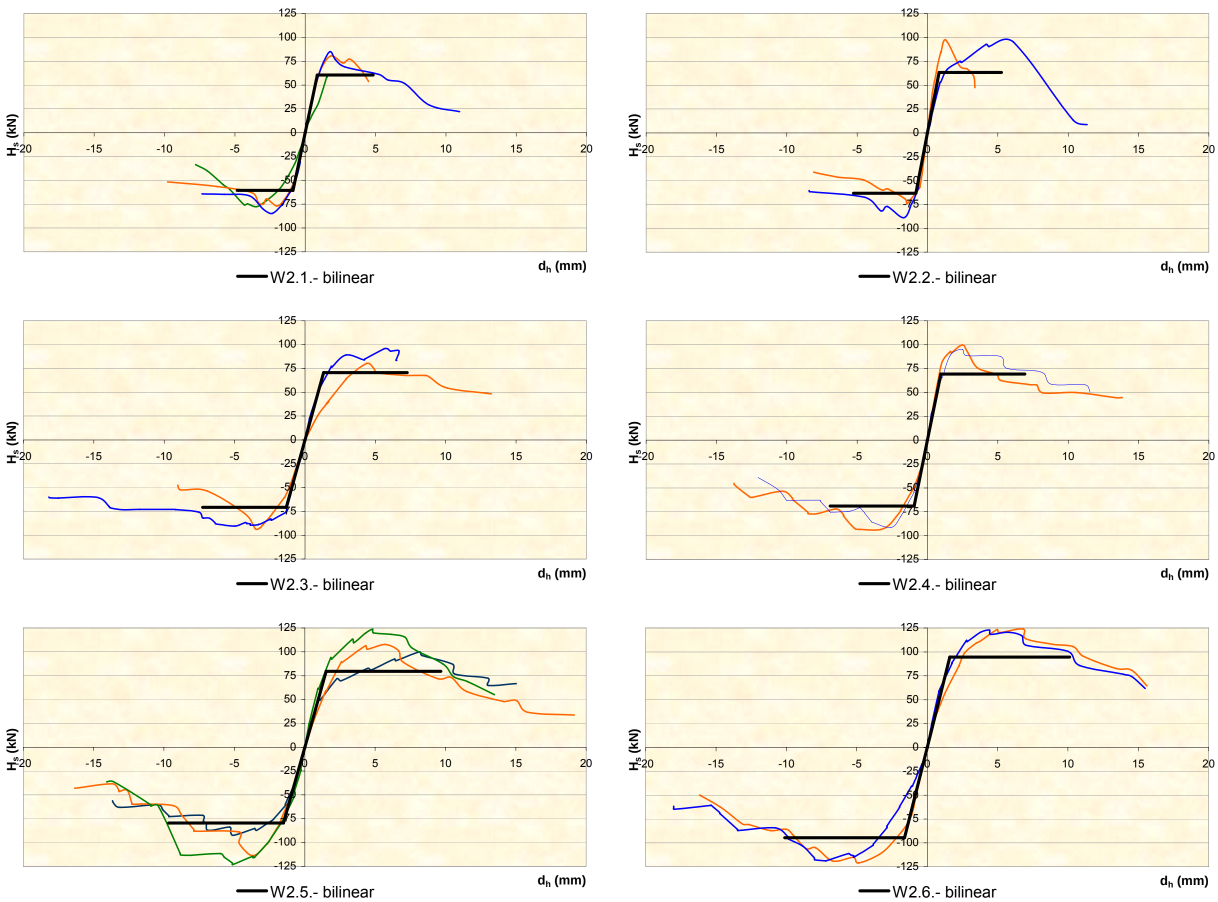

Figure 6. Envelope of experimental values and bilinear diagrams.

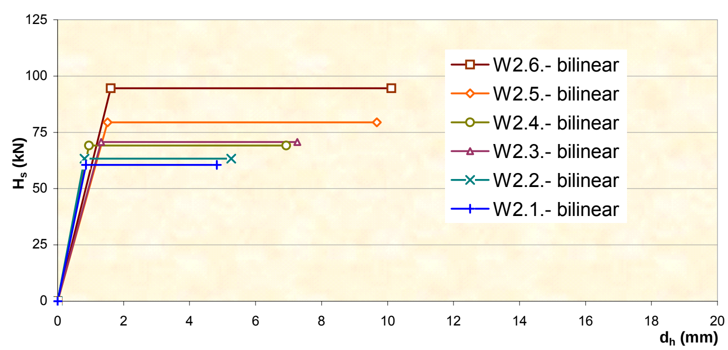

Figure 7. Comparison between bilinear diagrams, for the different masonry types.

Energy of Deformation and Stiffness

The behavior with respect to the dissipation of energy can be grouped in two groups: unreinforced masonry (W2.1, W2.2, W2.3) and confined masonry (W2.4, W2.5, W2.6), being the energy dissipation higher in confined masonry. In general, the dissipation of energy is very low in the first loading cycles, and the energy dissipation increases significantly with the appearance of diagonal cracking.

Table 4 presents compared values for the stiffness corresponding to the first crack, for the stiffness corresponding to the maximum load and the energy measured by the experimental envelope, allowing making the following observations:
- Vertical joint filling leads to marginally higher elastic stiffness and marginally lower stiffness at maximum load;
- The stiffness value is only marginally affected by the confining elements, even if confined walls possess a higher energy of deformation, associated with enhanced ductility;
- The addition of bed joint reinforcement allows to reach an increase of 60% in the energy of deformation.

Table 4. Comparison of energy of deformation and stiffness.

Comparison between wall groups	H_{cr} (–)	d_{cr} (–)	K_e (–)	H_{max} (–)	d_{Hmax} (–)	K_{Hmax} (–)	A_{real} (–)
filled / unfilled vertical joints							
W2.2/W2.1	1.27	1.15	**1.10**	1.10	1.16	**0.95**	**1.21**
confined wall / unreinforced wall							
W2.4/W2.1	1.14	1.11	**1.03**	1.17	1.36	**0.86**	**1.84**
W2.5/W2.3	1.19	1.22	**0.98**	1.22	1.15	**1.06**	**1.50**
average	1.17	1.16	**1.01**	1.20	1.26	**0.96**	**1.67**
effect of bed joint reinforcement							
W2.3/W2.1	0.91	1.20	**0.76**	1.15	2.07	**0.56**	**1.70**
W2.5/W2.4	0.95	1.31	**0.72**	1.20	1.76	**0.68**	**1.39**
W2.6/W2.4	1.08	1.33	**0.81**	1.28	1.89	**0.68**	**1.72**
average	0.98	1.28	**0.76**	1.21	1.90	**0.64**	**1.60**

Conclusions

Sixteen walls with different typology have been subjected to in plane lateral cyclic loading, keeping a constant vertical precompression level. The results allowed to assess the relevance of vertical joint filling, confining masonry elements and bed joint reinforcement. The difference in terms of strength was very moderate for the different configurations tested. In terms of deformation capacity and energy dissipation, the addition of confining elements and / or bed joint reinforcement represents a significant advantage. These two aspects are much more relevant that the usage of filled / unfilled vertical joints.

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