13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1093

EXPERIMENTAL BEHAVIOR OF MASONRY STRUCTURAL WALLS USED IN ARGENTINA.

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SUMMARY

Confined masonry is extensively used in seismic regions of Argentina. Experimental data about confined masonry built using local practice are very scarce and this lack of knowledge affects the seismic safety and the design practice of masonry structures. Some test results from real scale masonry models are presented in this paper. The models were designed based on the typical layout of a three story residential building and were built using the common practice in the Region of Cuyo in Argentina. Tests were performed for different displacement levels, with displacement control for lateral loads and load control for vertical loads. Six models of confined masonry walls with different column reinforcement were tested. Two of them had horizontal reinforcement placed at the joints. Behavior of the models during the test, as well the initial stiffness, strength, failure modes and plastic strain capacity are described in the paper. On the basis of the experimental results some conclusions are drawn in relation to strength and stiffness estimations by the Argentine building code, the influence of the vertical and horizontal reinforcement in the actual strength of the wall and the performance under severe earthquakes.

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INTRODUCTION

Confined masonry structures are convenient from an economic point of view. They are commonly used in the seismic regions of Argentina for buildings up to three floors. Normally this kind of building has masonry walls in two main perpendicular directions, joined by reinforced concrete slabs. Seismic action, represented by lateral forces applied to each floor and to the roof, is resisted by a mechanism of walls, coupled by lintels and sills, and connected by the slabs. Slabs are assumed to behave as non-deformable diaphragms, being able to distribute the lateral forces to the walls.

It is possible to estimate the theoretical flexural strength of a confined masonry wall in a simple way, by considering the amount of columns reinforcement, vertical load supported by the wall and yield stress of the reinforcement steel. On the other hand, the cracking shear load and the maximum shear strength of these walls are more uncertain, since they depend on several factors like: individual brick strength, mortar and workmanship qualities, vertical load, amount of columns reinforcement and amount of horizontal reinforcement embedded in the masonry. In addition, the manufacturing conditions of the bricks and the walls are very variable, causing high dispersion of the resulting mechanical properties.

DESCRIPTION OF THE TESTS

In order to obtain better knowledge about the seismic behavior of confined masonry walls used in the seismic region of Argentina tests were performed on six real scale model walls at the Earthquake Research Institute of the National University of San Juan (IDIA). The walls were built with handmade solid ceramic bricks, 18 cm wide. Wall confinement was provided by reinforced concrete columns, with nearly square sections, 20cm wide by the thickness of the wall. The design of the tested models was based on the typical building layout used by the San Juan Provincial Institute of Housing (IPV). The model dimensions are shown in Figure 1. The materials used for the walls were characterized by testing brick piers under simple compression and small masonry probes under diagonal compression, according to the Inpres-Cirsoc 103 Code [1]. A summary of the results of material characterization tests are presented in Table 1 [2]-[3]

Table 1. Summary of tests on brick piers and small walls.

Mortar (mixing ratio C:L:S)	σ_m [MN/m^2]	σ_c [MN/m^2]	τ_m [MN/m^2]
Normal strength (1:1:5)	4,1	2,7	0,22
Intermediate strength (1:1/2:4)	5,0	2,9	0,28
High strength (1:0:3)	8,7	6,1	0,31

- [σ_m]: Mean compression strength of piers.
- [σ_c]: Characteristic compression strength of piers.
- [τ_m]: Mean shear strength from diagonal shear tests.
- Solid ceramic bricks: mean compression strength 8.2 MN/ m^2, characteristic compression strength 4.5 MN/ m^2.
- Mean elasticity modulus measured in piers under simple compression: E= 1600MN/ m^2

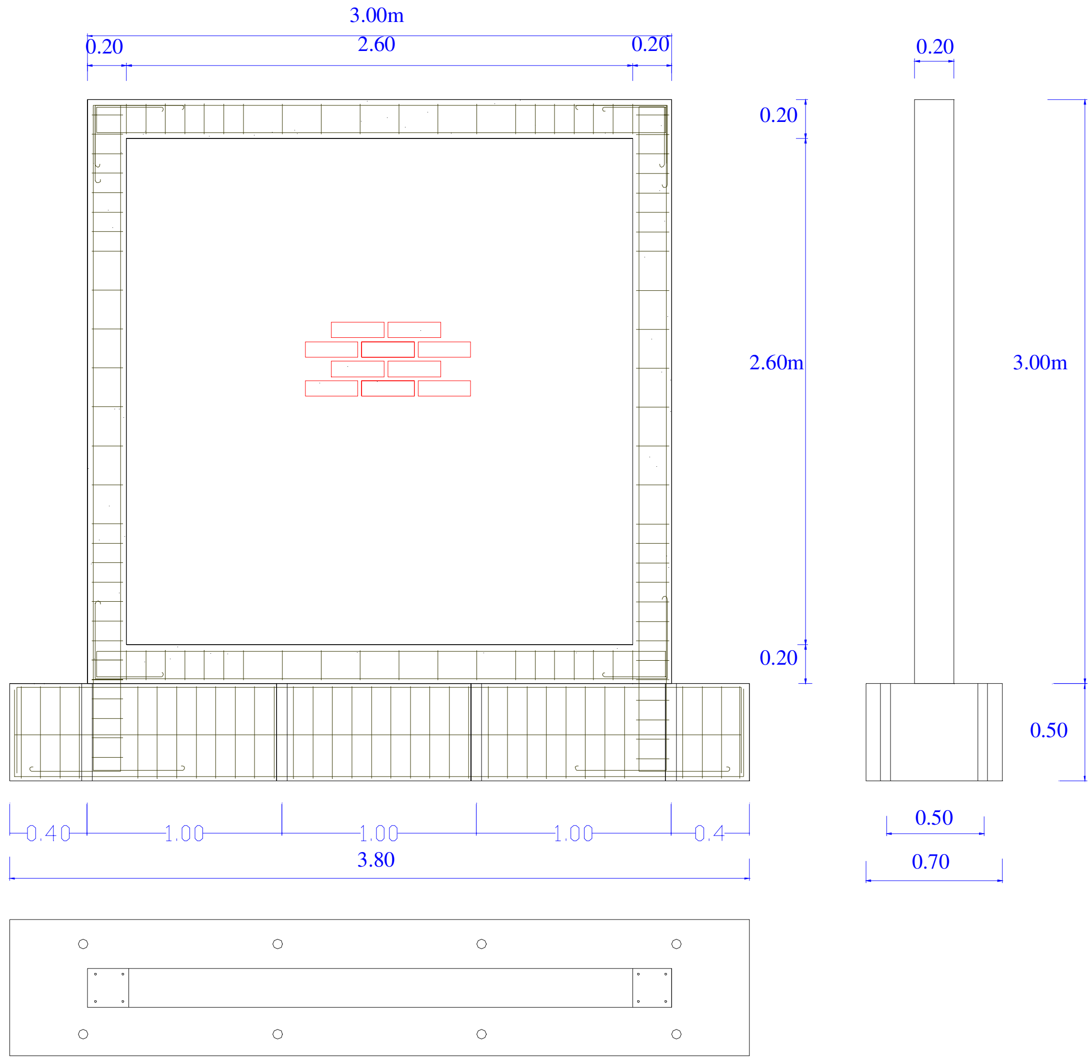

Figure 1. Model dimensions.

The walls were tested under a prescribed constant vertical load and allowing free rotation of the upper end. The vertical load was applied through a stiff steel beam by means of two vertical servo-controlled actuators. The tests were performed by applying cycles of lateral displacements at the wall head. An outline of the test setup and its instrumentation is presented in Figure 2. The instrumentation consisted of one displacement transducer controlling the horizontal displacement of the wall head (L.V.D.T.1), two vertical displacement transducers at both sides of the model (L.V.D.T 2 and 3), two diagonal displacement transducers (L.V.D.T. 4 and 5), three load cells mounted in series with the hydraulic jacks and a number of strain gages applied to some columns reinforcement bars.

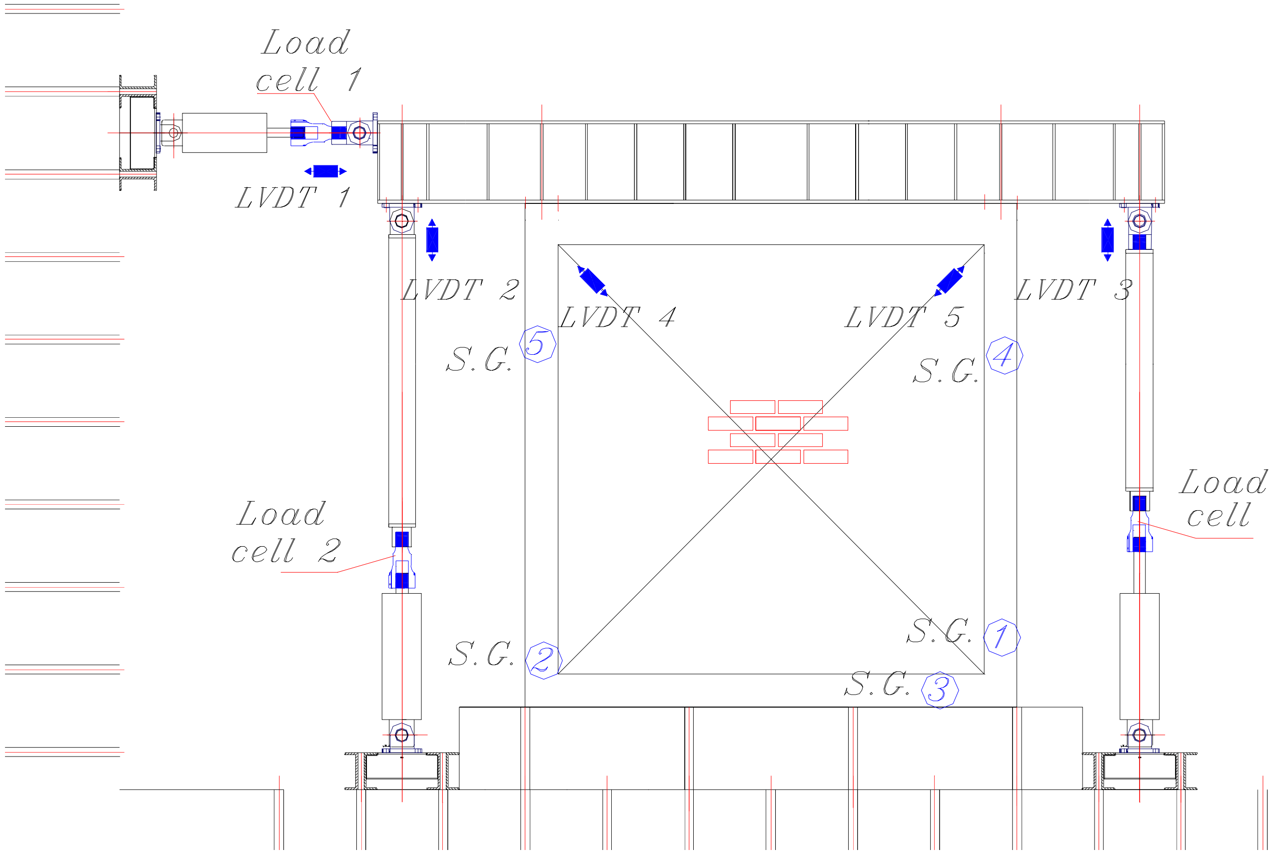

Figure 2.

ANALYSIS OF RESULTS

Table 2 summarizes the main features of the six tested walls: columns reinforcement, horizontal masonry reinforcement, vertical load, theoretical flexural capacity, estimated shear capacity using the expressions of the Inpres-Cirsoc 103 Code, and the maximum strength measured during the tests [4]. Values of theoretical flexural capacity, which vary with vertical load and columns vertical reinforcement, have been calculated considering the point of application of the horizontal load and the nominal yield strength of steel bars (420 MN/ m^2).

In walls 1 and 2, the amount of column reinforcement causes the flexural capacity to be slightly larger than the shear capacity estimated using the expressions of Inpres-Cirsoc 103 Code. In the case of the walls 3 and 4 the flexural capacity is several times larger than the shear capacity. In the third group of walls (5 and 6) the amount of vertical reinforcement was decreased and horizontal reinforcement embedded in the masonry was added, in order to ensure the shear capacity to be larger than the flexural capacity.

Table 2. Tested walls features and results.

Wall	Vertical reinforcement	Horizontal reinforcement	Vertical load. [kN]	Theoretical flexural capacity (1) [kN]	Estimated shear capacity. (2) [kN]	Maximum measured strength [kN]
1	4 ϕ10 (3.12 cm^2)	-	100	142	109	118
2	4 ϕ 10 (3.12 cm^2)	-	100	142	109	93
3	4 ϕ 16 (8.05cm^2)	-	200	342	138	207
4	4 ϕ 16 (8.05cm^2)	-	200	342	138	235
5	4 ϕ 8 (2.01 cm^2)	2 ϕ 6 each 2 mortar joint. (3.1 cm^2/m)	100	105	109+ 72 (3)	157
6	4 ϕ 8 (2.01 cm^2)	2 ϕ 6 each 2 mortar joint (3.1 cm^2/m)	100	105	109+ 72 (3)	169

Notes:
(1) Considering the horizontal load applied at the horizontal actuator level, the applied vertical load and σs= 420 MN/ m^2 (yield stress of the steel)
(2) Vur= (0.3 σ +0.6 τmo)[1]. Where σ = compression stress, τmo = diagonal shear strength of small masonry probes. τmo= 0.3 MN/ m^2
(3) Additional strength due to horizontal masonry reinforcement.

The models 1 to 4 developed the crack pattern presented in the sequence of Figure 7. This pattern includes diagonal cracking of the masonry panel and partial separation of the confinement columns. These walls clearly show a shear failure, but sustained their strength for a displacement up to 20 mm. Figure 6 summarizes the envelope curves of all tests. None of these walls reached their theoretical flexural capacity and the final state was controlled by the columns shear strength. This is due to the fact that, under large displacements, diagonal cracking of masonry extended to columns. Compression failure never occurred nor the emptying of the cracked masonry panel (See Figures 8 and 9).

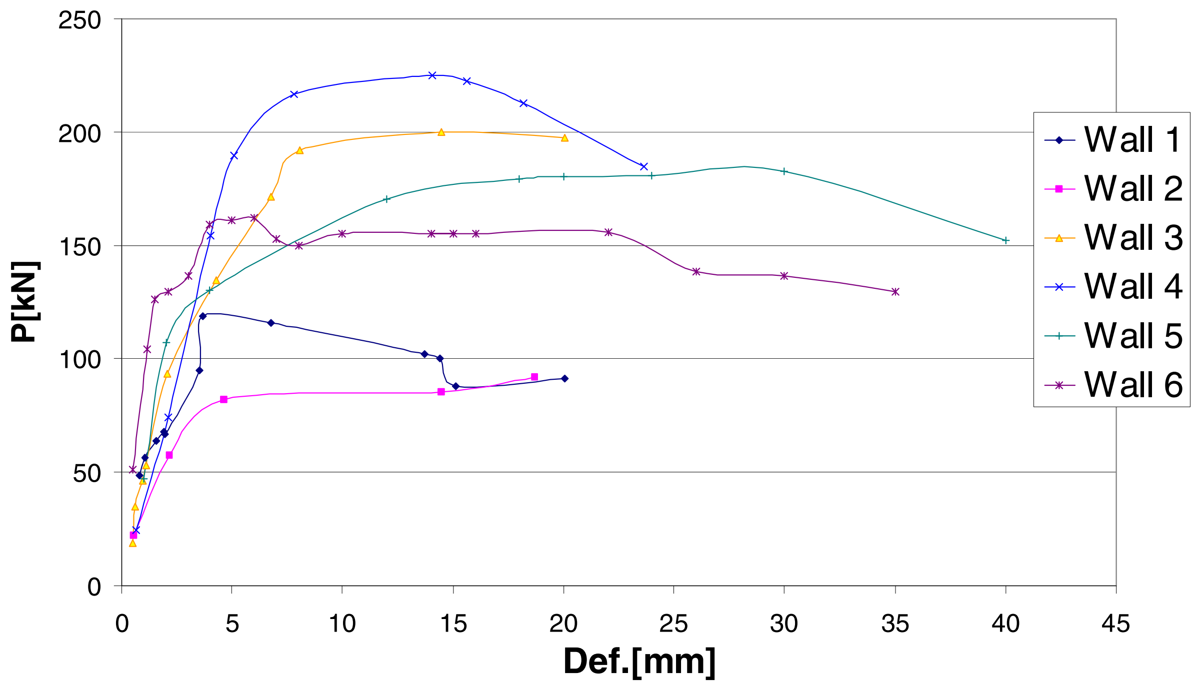

Figure 6. Load-displacement envelope curves.

The walls 1and 2 showed similar initial stiffness and cracking strength to those measured on walls 3 and 4. The latter sustained higher values of lateral load due to a combined truss frame mechanism in which diagonal compression of masonry is limited only by the shear strength of the confinement column, since vertical reinforcement bars do not reach yielding in tension. Flexural and shear capacities of the plastic hinge in the column are larger in these walls than in wall 1 and 2. In addition, greater columns-panel separation was observed in walls 3 and 4.

Walls 5 and 6, having a shear capacity clearly larger than the flexural capacity, reached, by hardening of the vertical reinforcement bars, strength values substantially larger than the theoretical flexural capacity. Under the applied displacement cycles, with increasing amplitude, it was observed that these walls (Figure 10) maintain their strength and their energy dissipation ability for larger displacement amplitudes than walls 1 to 4. Bending-induced horizontal cracking was observed and the separation between column and panel did not occur (Figure 12). The final state is controlled again by the shear strength of the column at the joints with the confinement beams. Figure 11 shows all the displacement cycles applied to wall 6. Each displacement amplitude was applied two times. It is possible to observe the stiffness degradation between the first and the second cycle with the same displacement amplitude.

The initial secant stiffness, measured at one per thousand drift, was higher for walls 5 and 6 than for walls 1 to 4. This is because the horizontal masonry reinforcement prevents the separation between column and the masonry panel. Table 3 shows the estimated stiffness by simple theory and the measured stiffness for 0.5 and 1 per thousand drift.

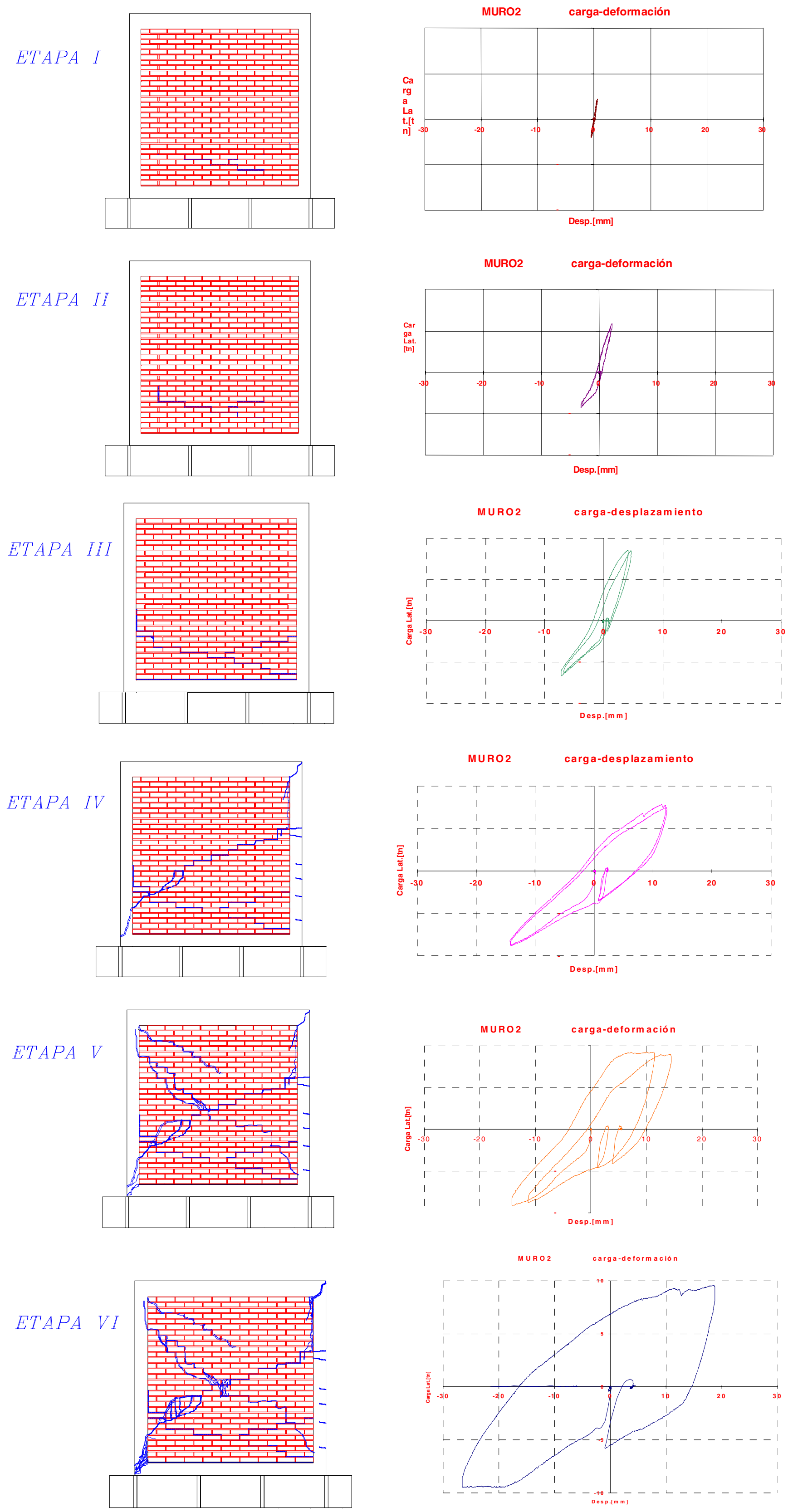

Figure 7. Damage sequence for wall 2.

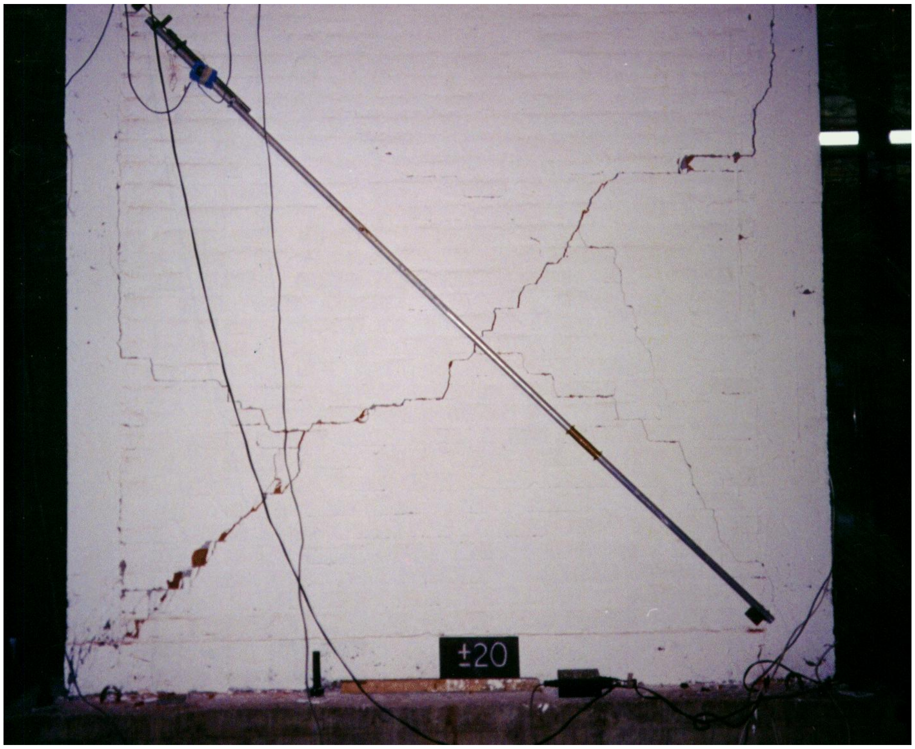

Figure 8. Final cracking for wall 4.

Figure 9. Shear failure at column in wall 1.

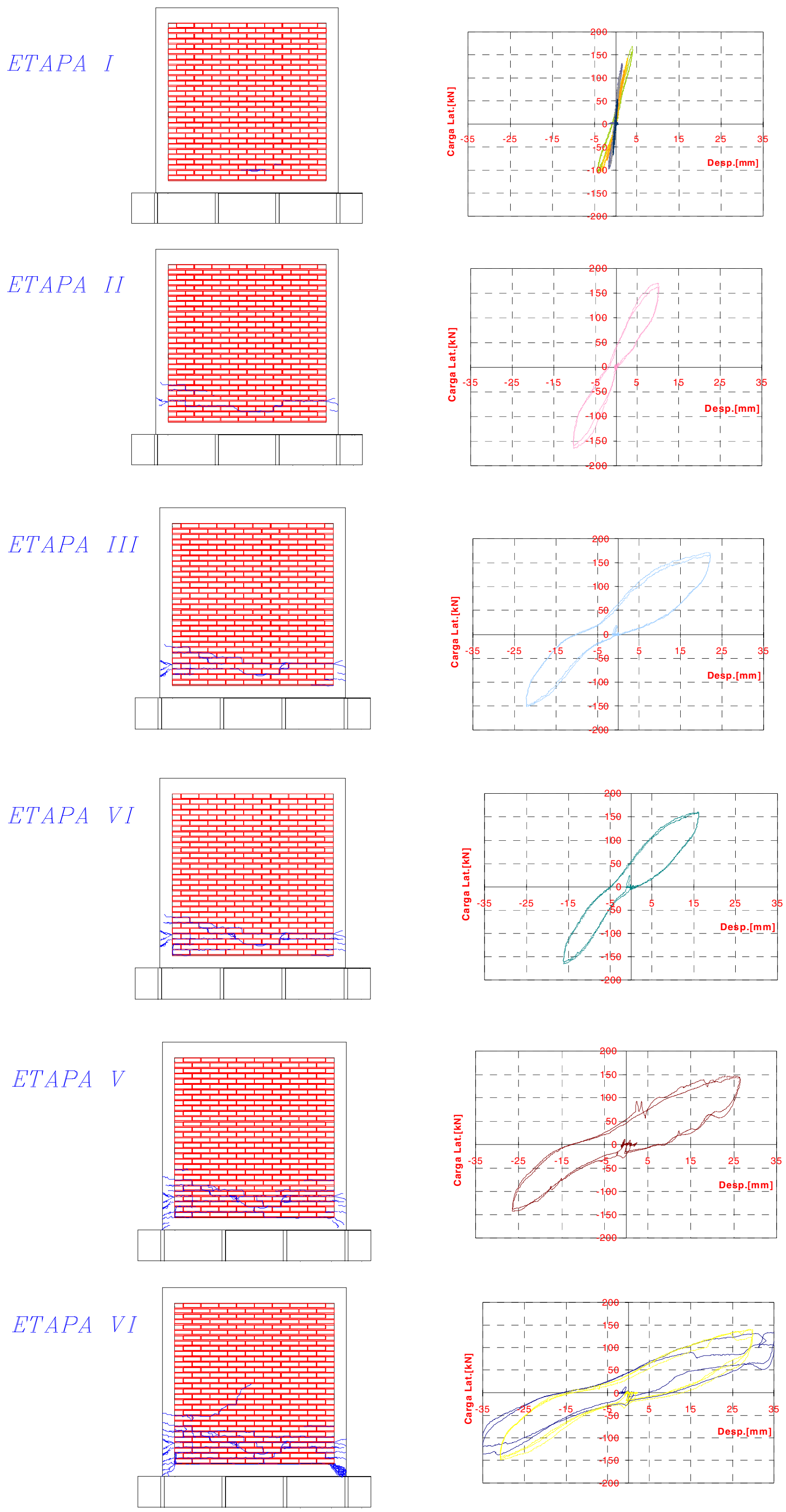

Figure 10. Damage sequence for wall 6.

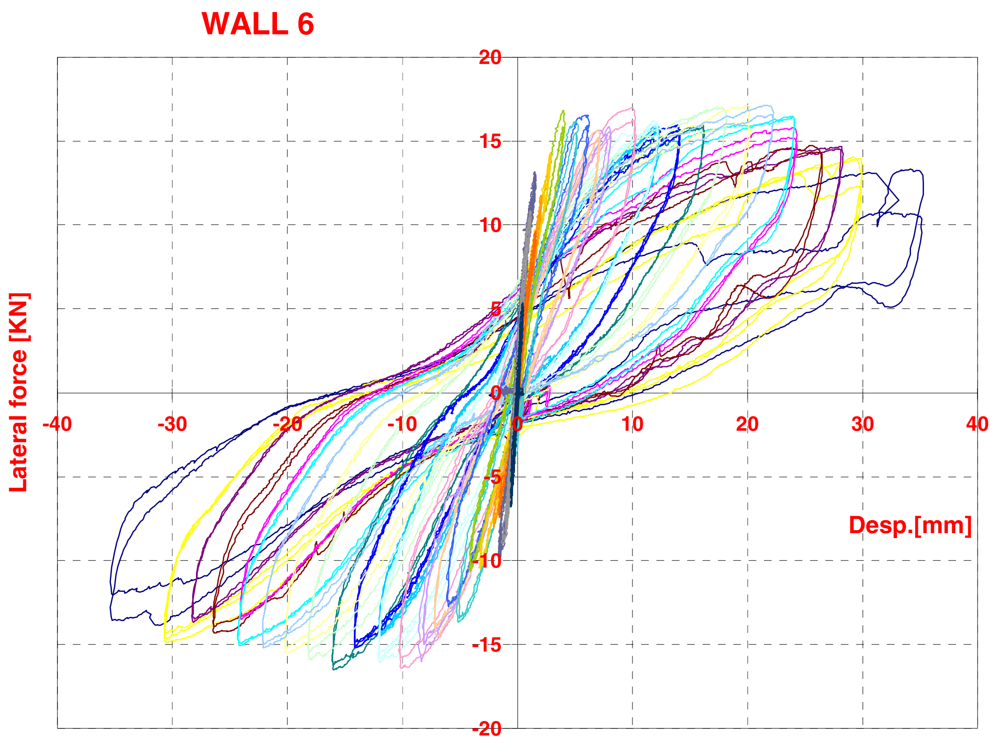

Figure 11. Wall 6 tests.

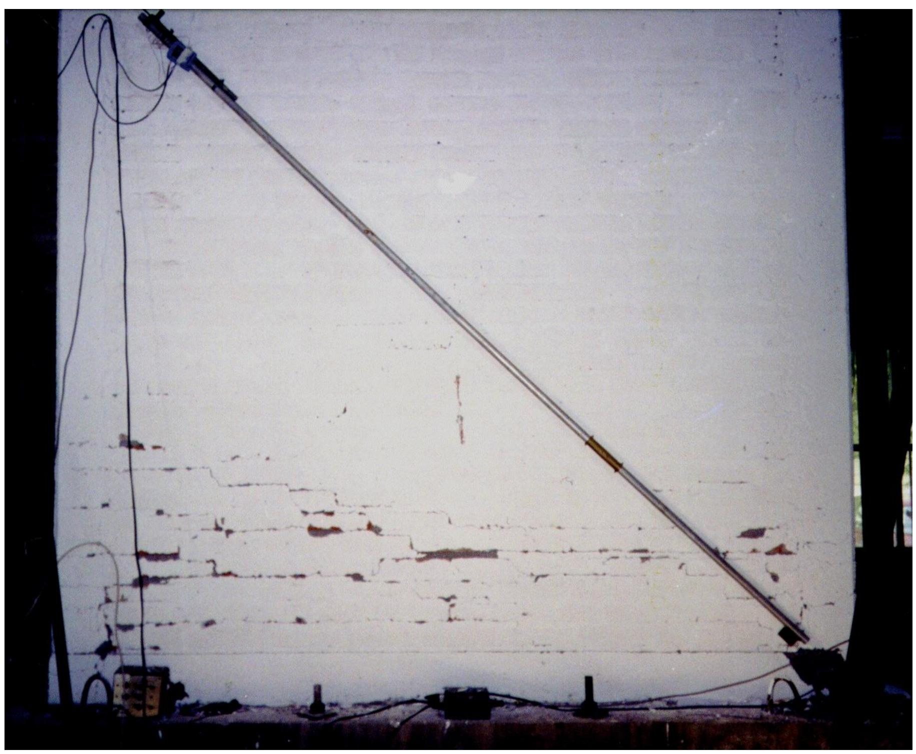

Figure 12. Final state of the wall 6.

Table 3. Estimated and measured secant stiffness

Wall	K $_e$[kN/mm]	K $_{0,5}{}^{0}/_{00}$[akN/mm]	K $_{1}{}^{0}/_{00}$[kN/mm]
1	$\dfrac{1}{\dfrac{h^3}{3EmJm}+\dfrac{h}{GmAm}} = 39$	52	39
2		31	23
3		50	37
4		52	45
5	$\dfrac{1}{\dfrac{h^3}{3EmJt}+\dfrac{h}{GmAm}} = 74$	72	47
6		83	54

h : wall height.
E_m=1600 MN/m2.Young modulus.
G_m=500 MN/m2. Shear modulus
A_m= Wall section without transformation of confinement columns sections.
J_m= Inertia moment without transformation of confinement columns sections.
J_t=. Inertia moment with transformation of confinement columns sections.
K $_{1}{}^{0}/_{00}$: Secant stiffness for 1 $^{0}/_{00}$ drift.

CONCLUSIONS

The Inpres-Cirsoc building code allows a reasonable estimation of the wall strength, based on measured strength in diagonal shear test of small masonry probes, only in the case of lightly reinforced columns, not providing larger flexural capacity than shear capacity (See Table 2). For larger reinforcement ratios of columns, the wall strength is controlled by the shear strength of the confinement columns and beam joints. The code should require the capacity design of columns and joints reinforcement, considering the maximum expected shear force induced by the compressed masonry strut, arising from the cracking pattern of the panel. For the used brick type, a compression failure of this strut is not likely to occur and therefore the wall strength becomes controlled by the vertical reinforcement of the columns. The amount of transverse reinforcement in critical zones of the confinement columns and beams normally used in practice is insufficient in order to sustain this shear force.

Placing horizontal reinforcement bars with enough cross section ratio (0.18 %) so as to control the diagonal cracking and to increase shear strength, together with low ratios of vertical steel, proved to radically change the failure mechanism of the tested walls and increased the initial stiffness and plastic strain capacity. The Inpres-Cirsoc Code should include the horizontal reinforcement in computing the shear strength of the wall. It must be noted that the reinforcement steel placed in the tested walls is slightly larger than the minimum recommended by the code (2 ϕ 4.2 c/50 cm).

The secant stiffness of walls without horizontal reinforcement, for a drift level of 0.001, can be estimated in a simple way by considering the uncracked gross section of the wall as homogeneous. On the other hand, for walls with horizontal reinforcement steel in which the confinement column do not separate from the panel, it is possible to estimate the initial stiffness by considering the transformed and cracked sections of the confinement columns.

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