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EXPERIMENTAL EVALUATION OF CONFINED MASONRY WALLS WITH SEVERAL CONFINING-COLUMNS

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SUMMARY

The aim of this paper is to present the results of four tests performed at the IMME to evaluate the effect of the number of vertical confining elements, called confining-columns in this paper, in the seismic behavior of confined masonry walls. The walls were tested under reversed cyclic lateral loads and constant vertical load. Four full-scale walls of the same nominal area were constructed containing two, three and four confining-columns: Specimen M1 consisted of one panel and two confining-columns; Specimen M2 consisted of two panels and three equally spaced confining-columns; Specimen M3 also consisted of two panels but the central confining-column was located at ⅓ of the wall length, and Specimen M4 contained three panels and four equally spaced confining-columns. The results obtained show how the number of confining-columns affects the stiffness degradation, the energy dissipation capacity, the ductility, the cracking pattern and the strength of the walls. These results will be useful to improve the recommendation for analysis and design of confined masonry wall structures to adequately withstand severe earthquakes.

INTRODUCTION

With the aim of reducing the risks involved in the use of load-bearing masonry while keeping it as an alternative type of building system, the Instituto de Materiales y Modelos Estructurales (IMME) at the Facultad de Ingeniería at the Universidad Central de Venezuela, undertook a research project with the participation of research professors, technicians and both undergraduate and graduate students, to provide guidelines for the use of this type of structure (López et al. [1]). The work described in this paper is part of that project.

Confined masonry is a structural system widely used for housing in Latin America, Europe and Asia. It consists basically of masonry panels and slender casted-in-place confining elements. The masonry panels are usually made of clay bricks, clay or concrete blocks, bonded with cement mortar. The confinement consists of vertical and horizontal confining elements usually of reinforced concrete, called in this paper confining-columns and top beams, respectively. The cross-section dimensions of the confining elements are comparable with the thickness of the masonry panels.

In technical literature a lot of information can be found about the experimental evaluation of confined masonry walls, composed of one masonry panel and two confining-columns, under seismic-like in-plane

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loading. For instance Hernandez and Meli [2] present and discuss the results of twelve tests on reinforced concrete masonry walls to obtain recommendation for design and detailing, Gallegos [3] reports the results of 22 tests on reinforced masonry under seismic loading, Castilla [4] presents a complete study on confined masonry with hollow clay blocks, Carrillo and Molina [5] present another study to assess the behavior of confined solid clay-brick walls, Tomazevic and Klemenc [6] present the results of tests and use them for modeling the behavior of confined masonry walls, Yoshimura et al. [7] show the results of eighteen tests on half scale specimens and Castilla and Marinilli [8] present a complete study on confined masonry with hollow concrete blocks.

However, little information can be found about the seismic evaluation of walls with three or more confining-columns: Alcocer et al. [9] study the behavior of three confined masonry walls with openings, each one composed of two confined masonry panels with several degrees of coupling in flexion between them, Liu and Wang [10] present some aspects of confined masonry walls with several horizontal and vertical confining elements, and San Bartolomé [11] reports the results of three half-scale clay-brick confined masonry walls with several square masonry panels tested at the Pontificia Universidad Católica del Perú at Lima.

Meanwhile, the presence of more than two confining-columns in a wall is very common in practice due to the limitation in the length of the masonry panels, the presence of openings in the walls like doors and windows, and the intersection of walls. Based on this, the experimental evaluation of walls with more than two confining-columns is of great concern to rationalize its analysis and design in earthquake engineering practice. The objective of this investigation is to evaluate experimentally the effect of the number and spacing of confining-columns in the behavior of confined masonry walls under in-plane seismic-like loads.

EXPERIMENTAL EVALUATION

Description of the specimens

A set of four (4) full-scale confined concrete masonry walls of the same nominal area was constructed at the IMME, to be tested against constant vertical load and reversed cyclic lateral load. The first specimen "M1" consisted of one panel and two confining-columns. The second specimen "M2" consisted of two panels and three equally spaced confining-columns. The third specimen "M3" also consisted of two panels, but the central confining-column was located at ⅓ of the specimen length. Finally, the fourth specimen "M4" contained three panels and four equally spaced confining-columns. Figure 1 shows the configuration of the four specimens. The length and the height of the specimen were 300 and 230 cm, respectively.

The basic components of the masonry walls were hollow concrete blocks, mortar and confining elements. The concrete blocks used to build the walls had nominal measurements of 40x15x20 cm (length, width and height). Three types of tests were used to determine the mechanical properties of the masonry: five (5) blocks were compression tested until failure with an average strength of f_p = 85.07 kgf/cm^2; five (5) axial compression piles with an average strength of f_m = 68.00 kgf/cm^2; and five (5) 100x100 cm wall segments tested for diagonal compression with an average shear of v_m = 5.11 kg/cm^2. All the results are referred to the gross area of the blocks. The mortar used had a volume ratio of 4:1:1 of sand, lime and cement. To determine the mechanical properties of the mortar five (5) cubes were compression tested until failure with an average strength of 70.10 kgf/cm^2. Figure 2 shows the process of construction of the specimens. It is important to note that in all the walls the vertical joints could be properly filled with mortar due to the characteristics of the pieces.

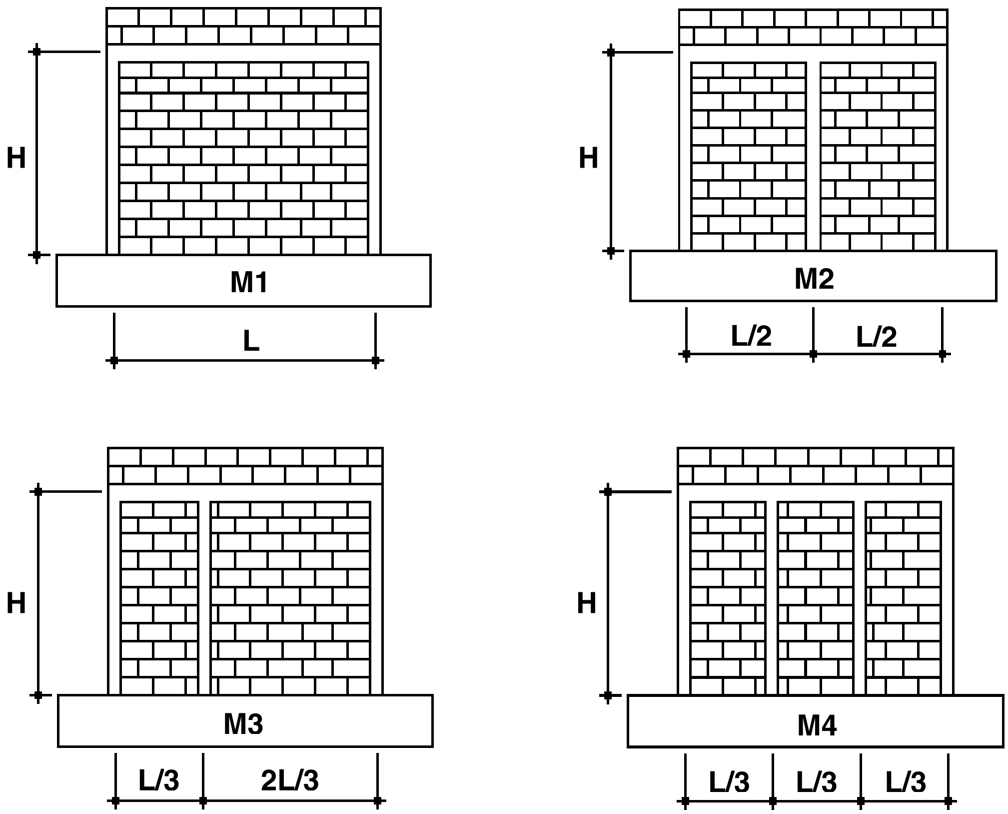

Figure 1. Configuration of specimens M1, M2, M3 and M4.

The confining elements were of reinforced concrete and consisted of a large 30x50 cm cross-section foundation beam, confining-columns of 15x15 cm and a top beam measuring 15x20 cm. The confining-columns and the top beam were reinforced lengthwise with four (4) No.4 bars. The confining-columns were reinforced transversally with No.3 stirrups at 6 cm intervals at the 40 cm ends of the elements and at 12 cm intervals in the remaining portions. The top beams were reinforced transversally with No.3 stirrups at 10 cm intervals. The concrete used had average strengths of 259 kgf/cm^2 and 238 kgf/cm^2 for the foundation beam and the other confining elements, respectively. The reinforcement steel used had an average yield stress of 4473 kgf/cm^2 , with a nominal yield stress of 4200 kgf/cm^2. Figure 3 shows the specimens finished.

Testing procedure

All the walls were tested at the Banco Universal de Ensayos of the IMME. The foundation beam for each wall was fixed to the floor of the *banco* using prestressed turnbuckles, so as to ensure that the specimen was properly anchored.

Each one of the walls was tested against lateral loads applied at the top of the wall. A steel box was placed around the top beam and fastened to it with bolts, so as to ensure an adequate distribution of the lateral loads along the wall. The loads were applied with alternating and increasing displacement-controlled cycles until the limit state of the walls was reached. Each cycle was repeated as many times as necessary to achieve stability in the corresponding hysteretic loop. The lateral loads were applied using hydraulic jacks. In addition each wall was subjected to a constant vertical load to simulate gravity effects. The vertical load was applied with a stiff steel girder and three dead weights, weighting in total 13.8 tf. To ensure a uniform distribution of the vertical load along the wall a sand bed was placed between the steel girder and the top of the wall. The vertical load applied represented a fraction of the strength of the piles (f_m) of 4.10 %.

Figure 2. View of the construction process.

Figure 3. Specimens finished.

The instrumentation employed consisted of two load cells to measure the load applied by the hydraulic jacks. Three LVDT to measure relative horizontal displacements between the wall and the foundation beam at three different heights (0.75, 1.50 and 2.30 m from the top of the foundation beam). A LVDT was placed at each confining-column to register its axial deformation during the test. While the test was being performed, a graph was made of the lateral load vs. relative lateral displacement at the top of the wall. All the information gathered was recorded and processed in a data acquisition system developed at the IMME. In each one of the specimens, the masonry was painted white so as to make it easier to observe the cracking of the walls during the tests. Figure 4 shows general features of the tests.

Figure 4. View of the tests assembly.

TEST RESULTS

Figures 5, 6, 7 and 8 show the specimens M1, M2, M3 and M4 after testing, respectively. Figures 9, 10, 11 and 12 show the hysteresis loops obtained during the tests for specimens M1, M2, M3 and M4, respectively. Table 1 shows the maximum displacements and loads obtained during the tests.

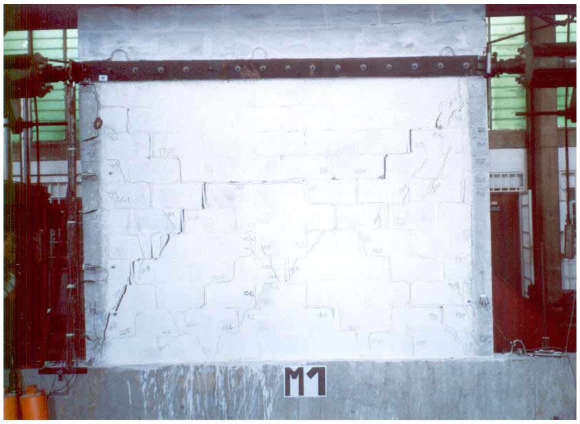

Figure 5. Specimen M1 after testing.

Figure 6. Specimen M2 after testing.

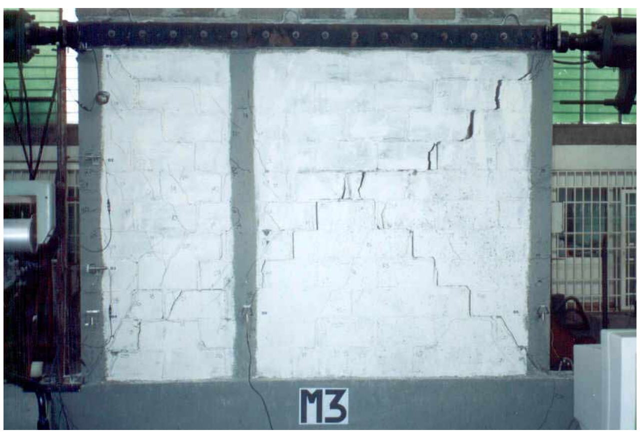

Figure 7. Specimen M3 after testing.

Figure 8. Specimen M4 after testing.

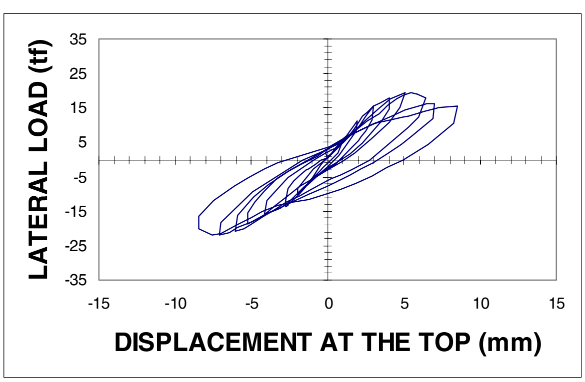

Figure 9. Hysteretic response of M1.

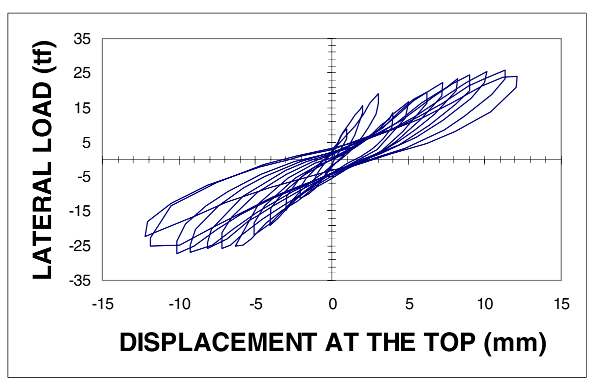

Figure 10. Hysteretic response of M2.

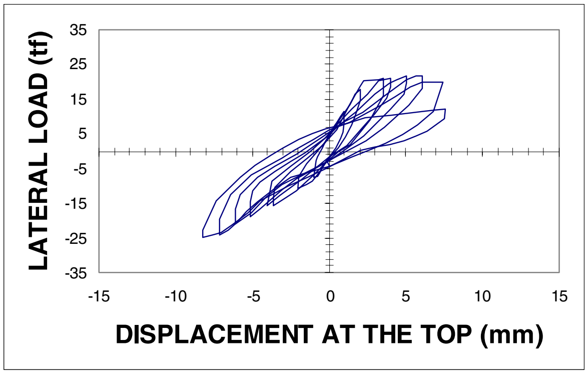

Figure 11. Hysteretic response of M3.

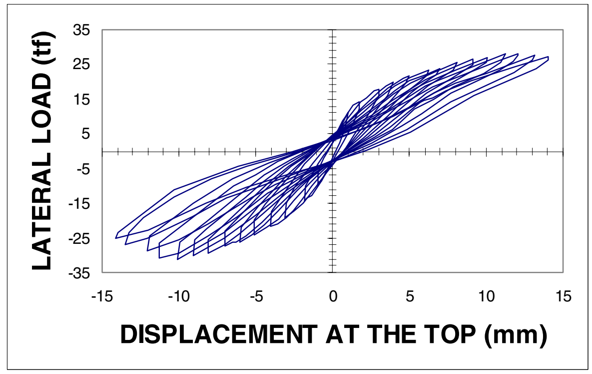

Figure 12. Hysteretic response of M4.

Table 1. Maximum displacements and loads obtained during the tests.

Specimen	Envelope	Maximum displacement (mm)	Maximum load (tf)
M1	Positive	8.534	19.401
	Negative	7.530	21.928
M2	Positive	12.113	25.793
	Negative	12.195	27.167
M3	Positive	7.587	21.702
	Negative	8.207	24.925
M4	Positive	14.077	28.054
	Negative	14.107	31.084

TESTS FINDINGS

Based on the hysteretic loops obtained at the tests, the positive and negative envelopes were obtained for each specimen. Also the secant stiffness, dissipated energy and equivalent damping coefficient for each of the cycles, and the elastic-perfect-plastic equivalent system with equal energy absorption, as described by Bertero [12], were found for each of the test walls. Based on this information, the yield displacement, the yield strength, and ultimate displacement for which there was no significant loss of strength were all calculated.

Stiffness degradation

When observing the hysteretic cycles in the four (4) walls, stiffness degradation caused by increased lateral deformation was found. Figure 13 shows the evolution of secant stiffness degradation during the tests.

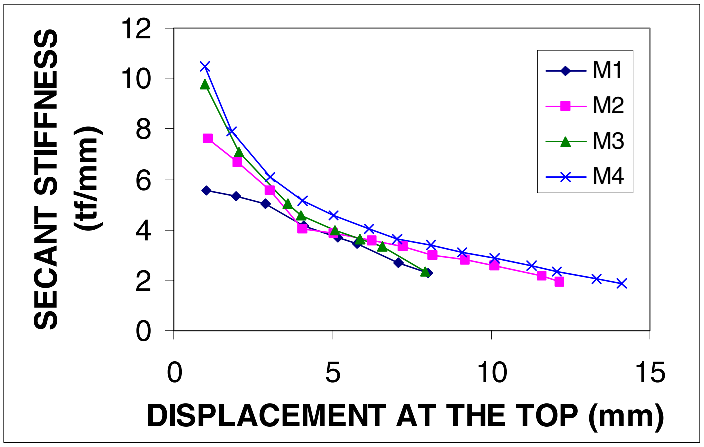

Figure 13. Stiffness degradation.

Energy dissipation
Figure 14 shows the energy dissipated per displacement unit by the specimens on each test cycle. The benefits of causing increasing damage to the walls with increased lateral deformation is reflected in a greater dissipation of energy.

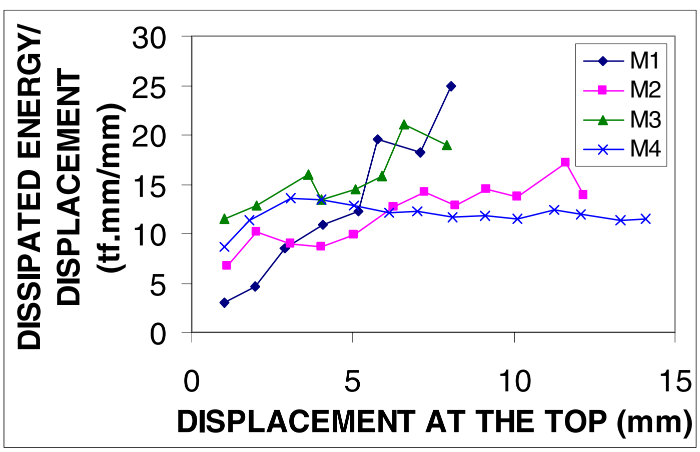

Figure 14. Energy dissipation.

Equivalent damping ratio
Figure 15 contains an equivalent damping ratio based on the energy dissipated by each hysteresis loop (Chopra [13]).

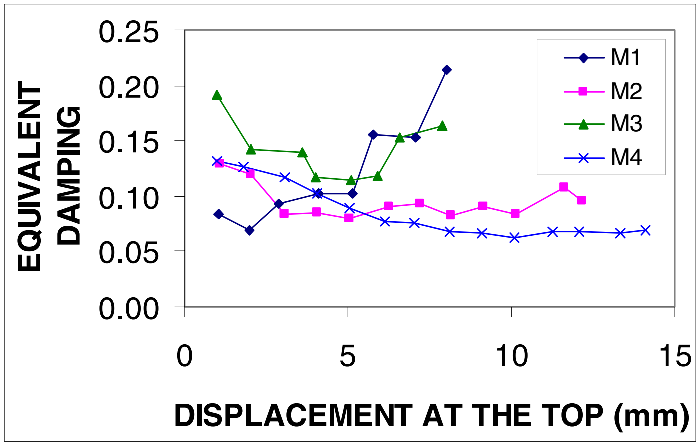

Figure 15. Equivalent damping ratio.

Elastic-perfect-plastic equivalent system

Table 2 contains the parameters of the elastic-perfect-plastic equivalent systems with equal energy absorption, as described before.

Table 2. Properties of the systems.

Specimen	Yield Displacement (mm)	Ultimate Displacement (mm)	Yield Strength (tf)	System Ductility	Equivalent Ductility
M1	4.838	8.032	20.665	1.66	1.50
M2	5.761	12.154	26.480	2.11	1.28
M3	5.481	7.897	23.314	1.44	1.29
M4	5.373	14.092	29.569	2.62	1.26

Equivalent ductility

The equivalent ductility is that obtained from an elastic-perfect-plastic equivalent system which dissipates the same energy as the cycle analyzed (Bertero [12]). Figure 16 shows the equivalent ductility against the system ductility for all the tested specimens.

Deformations

Figures 17 and 18 show as examples the lateral deformation of Specimens M1 and M4, respectively. These deformations were obtained with the LVDT used to measure the relative horizontal displacements between the wall and the foundation beam at three different heights.

Figures 19 and 20 show as examples the displacements recorded with LVDT4 (confining-column of the right in Figure 7) and LVDT6 (intermediate confining-column in Figure 7) during testing of specimen M3. Positive lectures of LVDT indicate lengthening of the confining-columns (tension field) while negative lectures indicate shortening (compression field).

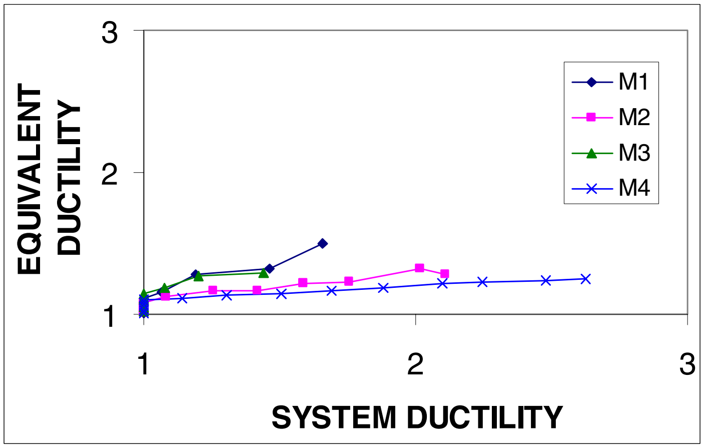

Figure 16. Equivalent ductility vs. System ductility.

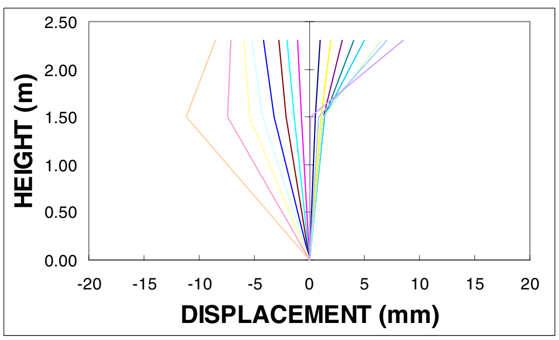

Figure 17. Deformation of specimen M1.

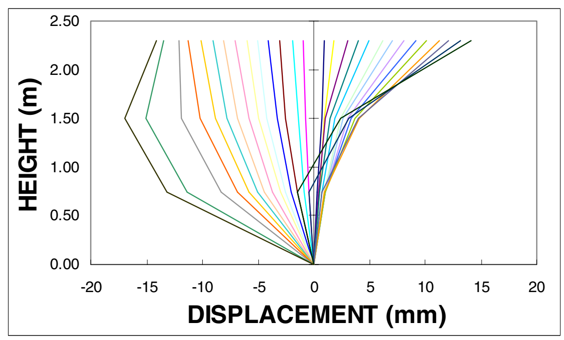

Figure 18. Deformation of specimen M4.

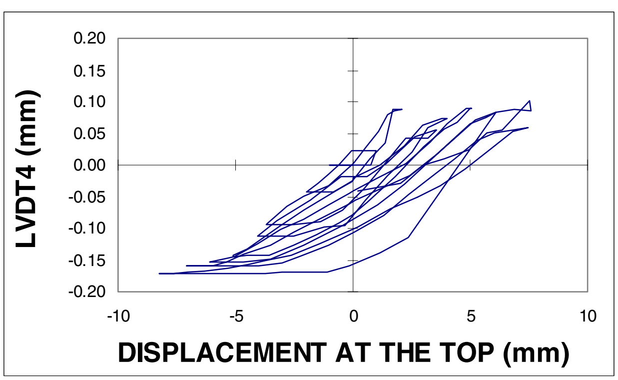

Figure 19. Displacement at LVDT 4 for M3.

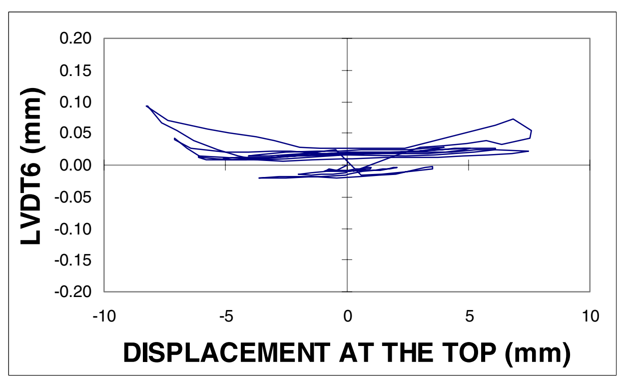

Figure 20. Displacement at LVDT6 for M3.

Cracking

In Figures 5, 6, 7 and 8 can be seen the cracking produced during testing in Specimens M1, M2, M3 and M4, respectively. Graded 45º cracking was found in all the masonry panels. The cracks even propagated to all the confining-columns.

Strength
Table 2 contains the strengths (yield strengths) obtained from all the tested walls.

DISCUSSION

Analyzing the initial secant stiffness of the walls in Figure 13, it is evident that Specimen M4 showed the biggest one, Specimen M1 showed the smallest one, while Specimens M2 and M3 showed intermediate ones. As can be predicted, the presence of more confining-columns in walls of the same global dimensions increases the initial stiffness of the walls. However, it is important to note that after stiffness degradation process during the tests, all the specimens showed similar residual stiffness at the end. The property of stiffness degradation has one important advantage from the standpoint of earthquake behavior, as those systems are able to dissipate energy without having to reach yield strength.

Specimens M1, M2 and M3 showed a tendency to increase the energy dissipation capacity for larger deformations, as can be seen in Figure 14. However, in the same figure it can be seen that Specimen M4 showed at the firsts cycles an increment in the energy dissipation capacity, but after that showed a tendency to keep, even reduce, its energy dissipation capacity. Based on this, the presence of more confining-columns does not seem to improve the energy dissipation capacity of the walls. Otherwise, it is necessary to insist on confined structural masonry and its actual ability to dissipate energy. This argument is important because, in general, design standards for seismic-resistant structures tend to allow the reduction of seismic actions due to inelastic response. Implicit in this reduction is the fact that the system has properties for the stable dissipation of energy, a condition that is generally achieved by means of proper hysteretic behavior of the materials. The test walls showed very little energy-dissipation capacity, although overall behavior showed a non-linear response to the lateral loads. This dissipation of energy was generated mainly by the friction occurring in the horizontal mortar joints. The experiments with these walls showed clearly that no earthquake-resistant energy-dissipation properties were found in this type of masonry.

Equivalent damping ratio showed at first a tendency to decrease for all the specimens, as can be seen in Figure 15. After that, Specimen M1 first, and Specimen M3 then, showed a tendency to increase the equivalent damping. However, for Specimens M2 and M4 equivalent damping ratio showed a tendency to be independent of displacement. Based on the results obtained it can be said that equivalent damping ratio generally ranges between 7% and 12% of the critical.

The system ductility registered during the tests ranged between 1.44 for Specimen M3 (1.66 for Specimen M1) and 2.62 for Specimen M4. Specimen M2 showed an intermediate value of 2.11. It results evident that the inclusion of more confining-columns improve the ability of the walls to make larger incursions in the inelastic range. This can be explained considering that less spaced confining-columns are able to perform a better confinement of the masonry panels. Otherwise, the behavior of equivalent ductility runs in the opposite way, since Specimen M1 shows an equivalent ductility of 1.50 and Specimen M4 of 1.26. Specimens M2 and M3 show similar values of 1.28 and 1.29, respectively. To understand this point it must be remembered that equivalent ductility is strongly dependent on energy dissipation capacity.

The analysis of the deformations obtained during the tests shows that the general behavior of the walls was governed by shear deformations, even for the specimens which deformations are not shown herein. The analysis of Figure 19 shows that when the wall is deformed to the right the right confining-column is in tension and when the wall is deformed to the left the right confining-column is in compression. The opposite behavior was observed in LVDT5 (confining-column of the left in Figure 7), even not shown here. Meanwhile, the intermediate confining-column performs basically in tension, as can be seen in

Figure 20. The discussed behavior was similar in all the tested specimens and it is consistent with the anticipated resistant mechanism for overturning moment.

The cracking observed in all the specimens occurred primarily along the horizontal and vertical mortar joints, following a 45° inclination due to the size of the blocks and the offset used in building the walls. The low level of vertical load applied during the tests was unable to change this crack pattern. In all the cases the cracking propagated to all the confining-columns. However, there are some facts to highlight: Specimen M1 suffered a horizontal crack along a mortar joint; Specimens M2 and M4 showed a widespread cracking distribution and Specimen M3 showed in the largest masonry panel a cracking pattern similar to that observed in M1, while the cracking at the shortest panel was similar to that observed in M2 and M4. Based on the results obtained, it can be said that the presence of more confining-columns at a lesser spacing seems to spread the cracking along the masonry panels, thus improving the damage distribution.

The values contained in Table 2 show that the inclusion of confining-columns tends to increase the strength of the walls. It is important to remember that all the tested walls had the same nominal transverse area and were tested against the same vertical load.

CONCLUSIONS

The effect of the number and spacing of confining-columns in the seismic behavior of confined masonry walls was evaluated experimentally in this paper. The results show that the inclusion of confining-columns in walls of the same nominal transverse area increases the initial stiffness, increases the system ductility, allows a better damage distribution in the masonry panels in conjunction with a lesser spacing of the confining-columns, and tends to increase the strength of the walls. Otherwise, the inclusion of confining-columns does not seem to improve the energy dissipation capacity or the equivalent damping ratio, and decreases the equivalent ductility of the walls.

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