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INFLUENCE OF HORIZONTAL REINFORCEMENT ON THE BEHAVIOR OF CONFINED MASONRY WALLS

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ABSTRACT

In order to assess the effectiveness of several reinforcing techniques for improving the seismic behavior of confined masonry walls, an experimental research program was undertaken. Six full-scale specimens were tested under alternated cyclic lateral loads. Specimens were tested to failure with increasing maximum drift ratios while a gravity load was applied and maintained constant. Two different types of reinforcement were used: a prefabricated ladder type joint steel and deformed cold-drawn wires. Specimens horizontally reinforced with deformed wires showed a remarkably improved behavior characterized by a larger deformation capacity and a reduced stiffness decay at large drifts.

KEYWORDS

Full-scale test; seismic test; confined masonry; walls; clay brick; horizontal reinforcement; steel wires; crack pattern; deformation capacity.

INTRODUCTION

In Mexico, as well as in many other countries, low-cost housing projects are mostly constructed using traditional methods for confined masonry. Confined masonry consists of load-bearing walls surrounded by small cast-in-place reinforced concrete columns and beams, hereafter referred to as tie-columns (TC's) and bond-beams (BB's), respectively. The system is such that walls must resist both vertical and lateral loads. Tie-columns have a square section whose dimensions typically correspond to the wall thickness (120 to 150 mm). Similarly, BB's width is equal to the wall thickness and the depth is usually equal to 250 mm. Typically, both TC's and BB's have longitudinal reinforcement ratios, based on gross sectional areas, of 1,2%. Confined masonry is used in buildings up to five stories high. For such cases, the TC's have higher longitudinal reinforcement ratios to resist overturning moments. Tie-columns and BB's are intended to confine the masonry panel, thus enhancing both the wall deformation capacity and the behavior under alternated lateral loads (as compared with unconfined masonry panels), and to improve the connections with other walls and floor diaphragms. Floor systems generally consist of cast-in-place reinforced concrete slabs, but very often prefabricated units are used (such as prestressed concrete joists or planks).

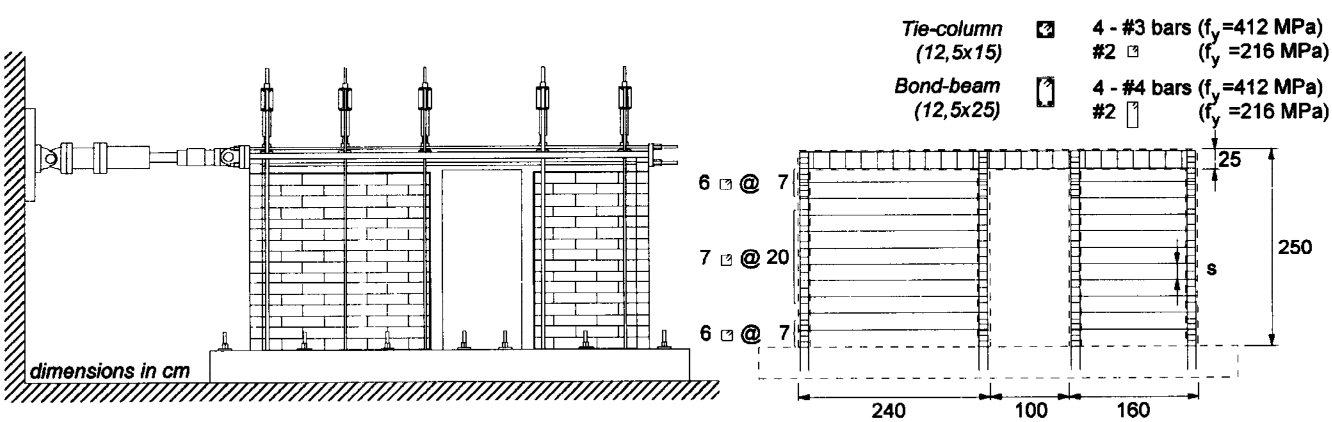
Confined masonry walls have limited shear strength and ductility as compared to common reinforced concrete walls; nevertheless, typical low-cost housing buildings have good earthquake resistance, because they have large wall densities (ratio of transverse wall areas to a typical floor area), and because wall layout is symmetric and regular, both in plan and in elevation. Their seismic behavior has been generally satisfactory, particularly in Mexico City. Nevertheless, significant damages have been observed in near-epicentral regions during strong ground shaking. New building code requirements in the country, enforced after the 1985 Mexico earthquakes, are more stringent than those of previous codes, thus requiring the designs to be revised, and in most cases, to be modified substantially to comply with the new code. Changes have led to the substitution of masonry walls by reinforced concrete walls, specially at the ground level and first story. Reinforcing alternatives aimed at increasing the shear strength of masonry walls have been evaluated. Small-diameter steel wires placed horizontally along mortar joints, and welded wire meshes anchored to wall faces and covered with cement mortar have been considered as feasible solutions.

To ascertain the seismic safety and to improve the design and reinforcement detailing of low-cost housing, an analytical and experimental research program is underway at the National Center for Disaster Prevention in Mexico (Alcocer and Meli, 1995). This paper deals with the phases aimed at evaluating the effect of the horizontal reinforcement on the behavior of walls subjected to alternated cyclic lateral loads. A thorough evaluation of test data can be found elsewhere (Díaz and Vázquez-del-Mercado, 1995; Aguilar, 1996).

SPECIMEN DESCRIPTION

Design Criteria and Fabrication

Specimens were designed and constructed following the requirements of the Mexico City Building Code (DDF, 1995). In series 1, models were two coupled walls (Fig. 1). Specimens were 5,0 m long; walls were 2,4 m and 1,6 m long, separated by a 1,0 m door opening. Structure height was 2,5 m. Specimen O had no horizontal reinforcement. The other two were reinforced horizontally within the running bond joints. Horizontal steel was calculated to maintain the predicted cracking load. In specimen E, a ladder type prefabricated reinforcement was used. It consisted of a set of two longitudinal high-strength cold-drawn smooth wires separated 90 mm by welded crosswires. Nominal wire yield strength was 491 MPa. Standard 10-gauge wire was used (ϕ3,41 mm). The ladder type steel was placed at every other joint (horizontal reinforcement ratio p_h equal to 0,102%). For specimen B, two high-strength deformed wires (ϕ4,0 mm) were spaced at one every third joint (reinforcement ratio equal to 0,091%). Nominal wire yield strength was 589 MPa.



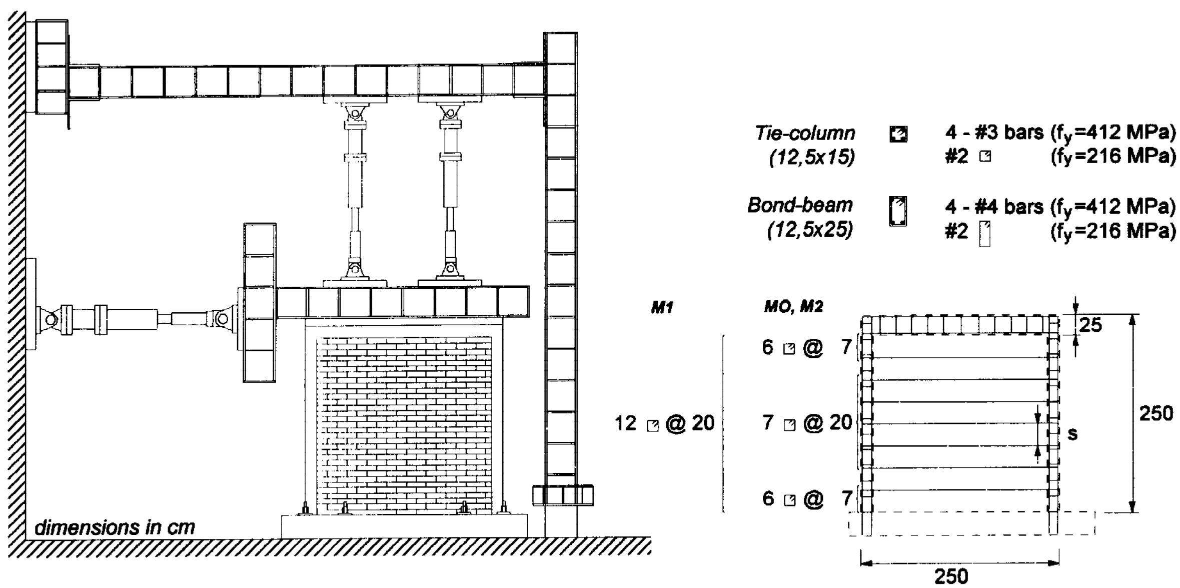
Specimen	Horizontal Reinforcement	Spacing, s [mm]	Horizontal reinforcement ratio, p_h [%]
O	none		0
E	ladder type prefabricated joint steel	140	0,102
B	2 high-strength deformed wires (ϕ4,0 mm)	210	0,091

Fig. 1. Specimens for series 1

In both horizontally reinforced specimens, bars were continuous along the wall and were anchored around the TC's longitudinal steel through 180-deg hooks bent in-situ after installation of the horizontal steel within the joint. From a constructibility point of view, specimen E was more easily fabricated than B. The position of the high-strength deformed wires in the wall thickness of B had to be corrected during construction.

In series 2, specimens were isolated square walls with 2,50 m side (Fig. 2). Main variable was the amount of wall horizontal reinforcement, which was made of wires of the same type as in model B. Horizontal reinforcement was anchored in the TC's by 90-deg hooks bent before placement. Specimen MO had not horizontal reinforcement; the ratio of horizontal reinforcement was 0,071% and 0,190% in the other two walls (specimens M1 and M2, respectively).

Walls were built with hand-made solid clay bricks. This material is typical for confined masonry in Mexico. Average brick dimensions were 250x125x62 mm for series 1, and 240x120x60 mm for series 2. The mortar used to join the bricks had a cement:sand ratio of 1:3.



Specimen	Horizontal Reinforcement	Spacing, s [mm]	Horizontal reinforcement ratio, p_h [%]
MO	none		0
M1	2 high-strength deformed wires (ϕ4,0 mm)	280	0,071
M2	2 high-strength deformed wires (ϕ6,4 mm)	280	0,190

Fig. 2. Specimens for series 2

Average compressive strength of mortar was 10,8 and 6,7 MPa for series 1 and series 2, respectively. For series 1 and 2, masonry prism strength was 5,4 and 3,5 MPa and diagonal compression strength was 0,52 and 0,25 MPa, respectively. The differences in masonry strength are attributed to the better quality of the mortar used in the first series and to its superior bond to the bricks.

Based on the hypothesis that a diagonal compression strut will eventually form within the masonry panel to resist the lateral loads, the following expression (Díaz and Vázquez-del-Mercado, 1995) was derived to calculate the maximum amount of horizontal reinforcement which could develop its total contribution prior to masonry crushing: $p_{h,max}=0.3f'_m/f_{yh}$, where f'_m and f_{yh} are the masonry compressive strength and the horizontal steel yield strength, respectively. The equation was obtained considering a 50% reduction of the prism compressive strength due to masonry inclined cracking, cyclic loading and elongation of horizontal wires. With this equation, maximum amounts of horizontal reinforcement were calculated for all specimens using measured material properties. Calculated $p_{h,max}$ are shown in Table 1. Reinforcement provided on M2 was equal to the maximum value associated to web crushing. Indeed, as it will be discussed later, M2 failed in shear-compression of the web.

Test Layout and Loading History

Test setups are illustrated in Figs. 1 and 2. Specimens were tested applying alternated cyclic lateral loads with a static-type hydraulic jack. To simulate the effect of gravity loading, a constant compressive stress equal to 0,49 MPa, which is considered typical for 4-to-5 stories low-cost housing buildings in Mexico, was applied during the test. Loading histories showed two phases. First, tests were load-controlled until reaching wall cracking load. In the second stage, which started at initial inclined cracking, displacement-controlled cycles with monotonically increasing drift ratios were applied to large drift levels (0,012 in series 1, and 0,020 in series 2). In this study, drift ratio was defined as the measured lateral displacement at the slab, divided by the wall height. To assess the stability of model behavior, two cycles at each drift ratio level were applied. In series 1, the formation of an inclined strut in the masonry panel due to the way of application of loads was forced. To better simulate the load application during earthquakes, in series 2 a stiff girder was used to distribute the lateral load along the wall.

TEST RESULTS

Crack Patterns

Final crack patterns for all specimens are shown in Fig. 3. Damage was characterized by inclined cracking extending through the wall panels. At failure, cracks penetrated into the TC's conducting to a sharp reduction in the lateral load carrying capacity of the specimens.

Cracks in specimens O, E and MO were concentrated along the diagonals, whereas structures B, M1 and M2 showed a much more uniform inclined cracking. For similar drifts, inclined crack widths of walls B, M1 and M2 were smaller than in E and on their corresponding control specimens (O and MO). At the end of the tests, horizontal wires of specimens E, B and M1 had fractured following a kind of a chain reaction. Once the first wires ruptured, stresses carried by this horizontal steel were redistributed thus increasing the deformations in the remainder wires and finally leading to their fracture. In contrast, horizontal reinforcement in M2 did not fracture. Brick crushing was observed at the center of the panel in specimen MO, and at midheight next to the TC's in M1 and M2.

At large drift ratios, wall deformation patterns in M1 and M2 showed relative horizontal movements of masonry blocks which followed the reinforced mortar joints (see Fig. 3). Near the wall toes, cracks were almost vertical and parallel to the TC's.

Walls with TC's transverse steel closely spaced showed less damage in TC's ends. In specimen M1, with TC's hoop constant spacing equal to 200 mm, failure was triggered by cracking and crushing of TC concrete and by bending of the longitudinal bars (bar kinking). In other walls, strength was maintained for larger drifts after TC cracking due to the lower spacing of TC hoops. The lower the hoop spacing the higher the bending resistance (dowel strength) of longitudinal bars and the larger its contribution to wall strength. The effect of the different fixtures for applying the lateral loads is apparent when crack patterns of O and MO are compared.

Hysteresis Curves

The story shear versus drift ratio curves for all models are also shown in Fig. 3. Graphs are drawn to the same scale to allow comparison among the specimens.

Hysteresis loops for specimen E were very similar to those of O. Elastic cycles showed some hysteresis attributed to wall flexural cracking at initial stages. Rounding of the loading branches in cycles to 0,003 was mainly associated to yielding of the horizontal reinforcement. Maximum shear forces exceeded code-calculated capacities up to the first cycle to 0,006 drift. Strength decay in further cycles to 0,006 and to 0,012 is associated to the brittle failure of longitudinal wires in the horizontal reinforcement. Comparing E and O, it is clear that the ladder-shaped joint reinforcement did not substantially improve the behavior. This observation is in agreement with the damage patterns and failure mode recorded. Similarly to specimen E, first cycles for B showed some nonlinearity due to wall flexural cracking and yielding of TC's longitudinal steel. When loading to 0,003 and 0,006 drifts, loops exhibited rounding due to excursions of horizontal steel into the plastic range. Cycles to these drift levels had good energy dissipation characteristics credited to plastification of the horizontal steel, and to brick crushing and friction. Pinching of loops at 0,006 drift was mainly attributed to wall shear deformations. Strength of B was 50% larger than that of E. Severe strength decay was observed after horizontal wires fractured at drifts to 0,012. Failure mode was similar to that exhibited by O and E. The horizontal deformed wires used in B clearly improved the hysteretic behavior of the structure. In B, yielding of the longitudinal reinforcement of TC's was credited to flexural deformations due to the increase in shear strength produced by the horizontal steel. In O and E, in contrast, yielding was due to shearing of TC's after the formation of the masonry strut (bar kinking at TC ends).

Similarly to O, specimen MO showed a behavior typical of confined masonry structures with no horizontal reinforcement subjected to alternated cyclic loading. After an initial elastic shear-drift ratio relation limited to masonry inclined cracking, loops exhibited hysteresis attributed to further cracking and brick crushing. After reaching the wall strength at 0,005 drift, a severe capacity decay and loop instability were observed. The latter is explained by the degradation of the masonry panel, as well as shear distress. In contrast, models M1 and M2 exhibited remarkably symmetric and stable curves up to drifts considerably larger than those accepted by the Mexico City Building Code (0,003, approximately). In both structures, strength was reached at drift levels close to 0,007; after this peak, wall resistance was maintained almost constant with a slight decay. Moreover, at drifts to 0,017, strength of M2 had decreased by 17% on the average from the peak value. Pinching of curves for M1 and M2 are explained by shear deformation of wall panels and by the relative horizontal displacement of masonry blocks observed along reinforced joints (see Fig. 3).

Plastification of horizontal wires in M1 started at a drift ratio of 0,0025. Deformations in the plastic range caused the wires to resist higher forces thus leading to an increased strength with cycling. Last plastification was recorded at a drift ratio close to 0,010 (Aguilar, 1996). First plastification of horizontal wires for M2 took place after 0,010 drift cycles until failure. Delay in the horizontal steel plastification is credited to larger flexural deformations of the specimen.

Shear Strength

Mexican regulations for masonry structures give procedures for calculating the design shear strength of walls, which in fact are intended to predict the diagonal cracking load. Thus, in design, the increase in load between first diagonal cracking and peak is conservatively disregarded. It has been observed in previous tests that the cracking load is almost independent from the amount of interior reinforcement. Therefore, the code-predicted wall capacity only depends on the unit shear strength of masonry and on the compressive stress due to gravity loads.

Measured shear stresses in walls are shown in Table 1. Shear stresses at first inclined cracking were slightly affected by the amount of horizontal reinforcement. Higher cracking stresses in series 1 are explained by the superior masonry shear strength presented before.

The peak to cracking stress ratio was higher for horizontally reinforced specimens. In unreinforced walls, once the inclined cracking has formed on the diagonals, the strength stability or its increase depended on the shear strength and detailing of TC's. Specifically, dowel resistance of longitudinal bars, caused by bar kinking, is the main contributor to TC shear strength at large deformations. Since TC section and reinforcement amounts are low, a high shear strength cannot be expected. The contribution of the horizontal reinforcement to the maximum shear capacity was always less than that predicted by assuming that the yield strength of all horizontal wires could be added to the shear strength of the masonry panel.

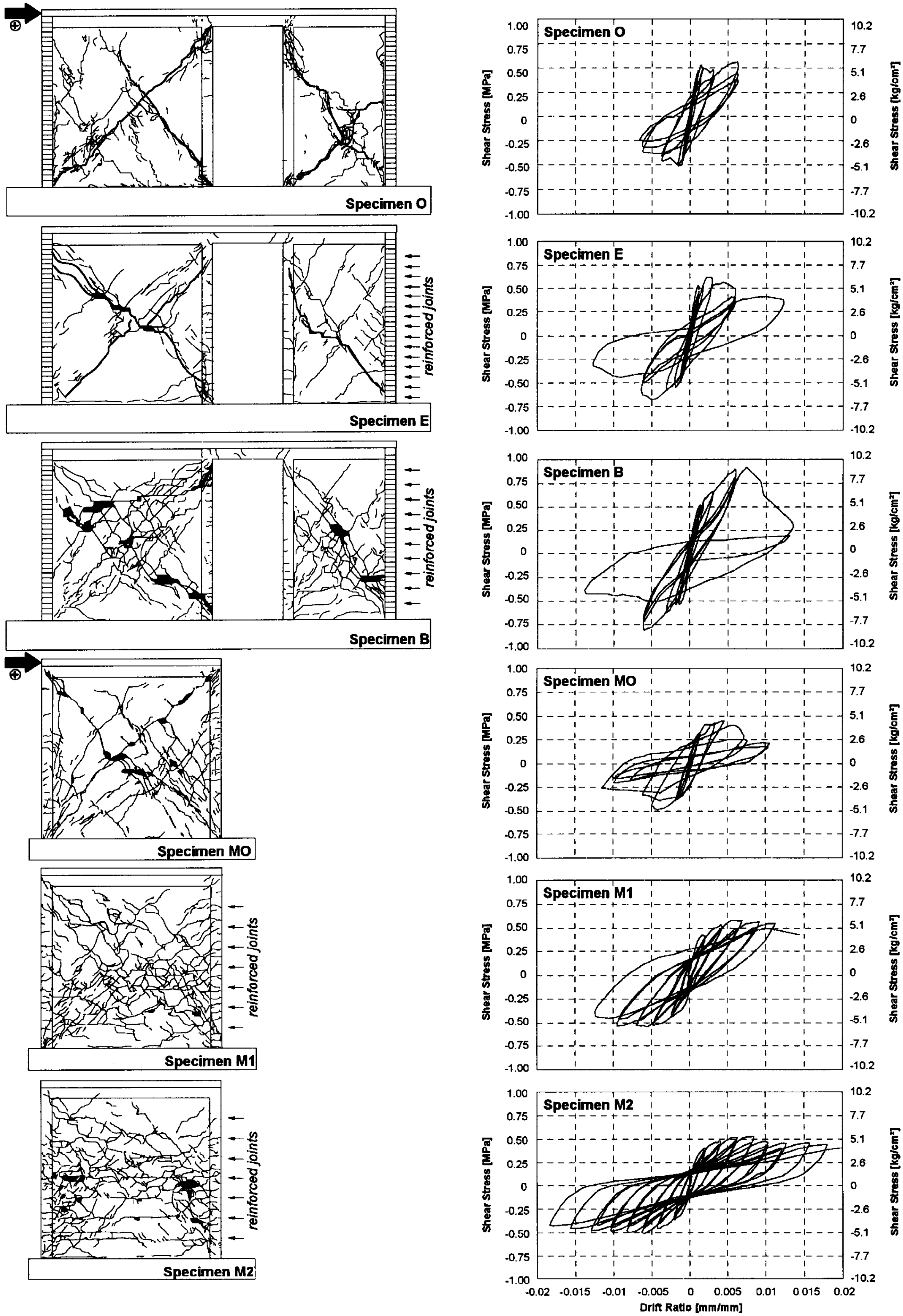


Fig. 3 Crack patterns and hysteresis curves

Table 1. Shear stresses for series 1 and series 2 specimens

Series	Model	p_h [%]	$p_{h,max}$ [%]	$p_h f_y$	Shear Stress: Code	Cracking	Maximum	Maximum / Cracking	η [%]	Ultimate drift ratio
	0	0,000		0,00	0,33	0,49	0,54	1,10		0,006*
1	E	0,102	0,408	0,52	0,31	0,53	0,61	1,15	40	0,005
	B	0,091	0,225	0,55	0,34	0,54	0,92	1,70	80	0,008
	M0	0,000		0,00	0,19	0,33	0,45	1,36		0,007
2	M1	0,071	0,155	0,43	0,24	0,43	0,58	1,35	70	0,010*
	M2	0,190	0,190	1,11	0,24	0,34	0,53	1,56	39	0,017

All values in MPa except those indicated

* *Taken from envelope curve as the maximum drift ratio applied*

To assess the participation of horizontal reinforcement to wall strength, an "efficiency" factor η was derived. This number was calculated as the load resisted by the wires divided by the nominal strength of horizontal wires. The load carried by each wire was obtained from strains measured in the test and converted to stresses through a measured stress-strain relation. The factor η reflects the non-uniform strain distribution of the steel reinforcement along the wall height. It has been verified that this distribution depends on the inclined crack width. In Table 1, the values η at wall strength are shown. Factors η were found to vary with the amount of reinforcement p_h and deformation level. The higher $p_h f_y$, the lower η. The higher $p_h f_y$, the higher are the loads and thus the deformations needed to mobilize the horizontal steel. However, the attainment of large η's at high load levels may not occur since prior masonry crushing may occur. The latter explains the low η-factor values recorded in M2 and the low shear stresses measured if the $p_h f_y$ values for other specimens are compared. As it was mentioned before, p_h for M2 was equal to $p_{h,max}$ calculated to avoid brick crushing.

Deformation Capacity

Load-deformation characteristics of the specimens can be studied through envelope curves (Fig. 4). As it was expected, curves greatly depart from the elastoplastic models commonly used to assess the inelastic behavior of structural members and their capacity to dissipate energy. None of the specimens showed a distinct yield point, and, in all cases, stiffness gradually decreased after first inclined cracking. Initial stiffness was similar for all walls regardless of the amount of horizontal reinforcement. The maximum strength was reached at drift ratios of approximately 0,005, and was greater for horizontally reinforced walls.

Since ductility factors may not be a representative measure of the inelastic behavior in this case, (because of the early non-linear behavior), a better measure of the deformation capacity is considered to be the ultimate drift ratio. This was defined as the drift at which 85% of the maximum load could be sustained. Values of this index are shown in Table 1. It is evident that ultimate drift for specimens with horizontal deformed wires were equal or higher than 0,008, whereas for unreinforced specimens the average value was 0,006.

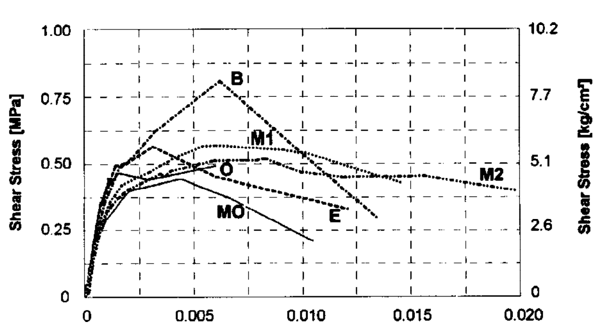


Fig. 4. Envelope curves

CONCLUSIONS

Specimens horizontally reinforced with deformed cold-drawn wires showed a considerably improved behavior under alternated cyclic lateral loads. Based on observations during tests and on analysis of data, the following conclusions have been obtained:

1. Crack patterns and failure mechanisms of specimens were dominated by shear deformations. Only one structure failed due to shear-compression distress in the masonry panel.
2. Horizontally reinforced specimens showed a more uniform distribution of inclined cracking as compared to the unreinforced control structures.
3. Theoretical capacity, calculated with current Mexican code equations, was exceeded in all specimens, particularly in those with horizontal reinforcement.
4. Use of deformed wires led to a substantial increase in strength and deformation capacity.
5. Horizontal reinforcement efficiency changed with wall drift level and amount of steel reinforcement.
6. Amount and type of horizontal steel had no apparent effect on specimens' initial stiffness. A parabolic tendency of stiffness degradation was similar to all specimens.
7. Horizontally reinforced specimens dissipated more energy than those with no reinforcement.
8. Due to the brittle mode of failure, a maximum drift ratio of horizontally reinforced confined masonry walls is considered to be 0,006.
9. Anchorage of horizontal steel by means of 90-deg hooks inside the confining elements had a satisfactory behavior during tests. Design bond stresses were not surpassed along the wires. The fabrication of this detail and placement of wires was easier than those with the 180-deg hook.
10. An equation to calculate the maximum amount of horizontal reinforcement based on plasticity assumptions and on the formation of a diagonal strut through the wall to resist load was developed. The maximum amount is intended to avoid wall crushing at drifts up to 0,01. Results using this formula agree well with test data.

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