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RETROFITTING OF CONFINED MASONRY WALLS WITH WELDED WIRE MESH

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ABSTRACT

The technical feasibility of jacketing (concrete mortar cover reinforced with steel welded wire meshes), as a rehabilitation technique for confined masonry walls, was assessed experimentally. Four full-scale specimens were rehabilitated and tested under alternated cyclic lateral loads. Variables were the level of damage (severely damaged and undamaged), type and size of specimens (two-story three-dimensional and one-story two-dimensional), the wire diameter of the mesh and the types of anchors used to attach the meshes to the masonry walls. Behaviors were compared to those observed in confined masonry structures with no reinforcement in the masonry panels, which were built as control specimens. Results indicate that jacketing of confined masonry walls with steel meshes and a mortar cover is an effective technique for improving the earthquake-resistant characteristics. A more uniform inclined crack pattern and a remarkably higher strength were observed in all specimens rehabilitated by jacketing as compared to the control masonry specimens.

KEYWORDS

Retrofitting; rehabilitation; repair/strengthening; confined masonry; walls; welded wire mesh; full-scale test; seismic tests; clay brick; crack pattern; shear strength; deformation capacity.

INTRODUCTION

Masonry has been the material most widely used for dwelling construction in Mexico. Typically, low-cost housing projects are built using traditional methods for confined masonry (CM) load-bearing walls. Main characteristics of confined masonry construction in Mexico can be found elsewhere (Alcocer and Meli, 1995; Aguilar *et al.*, 1996). Although the seismic behavior of Mexican confined masonry buildings has been, in general, adequate in far-epicentral areas (like in Mexico City), significant damages have been observed in near-epicentral regions during strong ground shaking. Damaged structures have been rehabilitated using different techniques. On the other hand, new building code requirements in Mexico, enforced after the 1985 earthquakes, are more stringent than those of previous codes. Thus, higher seismic load levels have forced designers to strengthen the load-bearing CM walls at the lower stories of a building. Wall jacketing with steel welded wire meshes and a concrete mortar cover is the most accepted technique among practicing engineers for repair and strengthening of damaged or undamaged structures.

Within the framework of a current research program on the structural safety of low-cost housing buildings at the National Center for Disaster Prevention, tests were performed aimed at assessing different schemes for repair and/or strengthening CM structures. This paper reports on the general results obtained from specimens rehabilitated (repaired/strengthened) by a jacket made of a cement mortar cover placed on the wall surface and reinforced with steel welded wire meshes (SWWM's). Structures were tested applying a constant vertical load and alternated cyclic lateral loads which simulated seismic-induced forces (Ruiz, 1995; Pineda, 1996).

RESEARCH SIGNIFICANCE

Although wall jacketing is widely used in Mexico, little experimental work has been conducted to verify its performance and its suitability as a rehabilitation technique. In Phase A, a damaged two-story full-scale three-dimensional CM wall structure was repaired with SWWM's and retested to failure (Ruiz, 1995). In Phase B, three isolated full-scale square walls, with no damage, were strengthened with SWWM's (Pineda, 1996). The size of the SWWM and the type of anchorage were varied in the specimens.

Other objective of this project was to investigate if wall jacketing might be a feasible option for increasing the shear strength and/or deformation capacity of new CM walls. If good seismic behavior were to be achieved with this reinforcing scheme, jacketed CM walls might substitute the reinforced concrete walls presently used at the lower stories of apartment buildings four-to-five stories high. Different reinforcing options have been assessed for improving the behavior of CM walls; one of the most promising ones is placing horizontal wires along mortar joints (Aguilar *et al.*, 1996). However, the use of SWWM's has the advantage that on-site supervision during construction is easier, and thus, the system behavior becomes more reliable. Nonlinear dynamic analysis of typical CM housing buildings are underway to further study the effect of the reinforcing options in the seismic response (Flores and Alcocer, 1996).

SPECIMEN DESCRIPTION AND REINFORCEMENT DETAILS

Phase A. Rehabilitation of a Two-Story Three-Dimensional Damaged Specimen

Structure dimensions are shown in Fig 1. Specimen 3D was designed and detailed according to present code requirements for masonry construction in Mexico City (DDF, 1995). The structure was designed to fail in shear at the ground floor to reproduce the failure mode observed in post-earthquake evaluations. The specimen consisted of two parallel wall systems of hand-made clay bricks; per level, each system had two panels with aspect ratios of 1 and 1,5. Panels were coupled by a cast-in-place reinforced concrete bond-beam/slab system. Average brick dimensions were 240 x 125 x 63 mm. Mortar cube strength was 16,2 MPa. Average masonry strengths were 5,3 MPa in axial compression and 0,59 MPa in diagonal compression. Reinforcement details of the original structure are shown in Fig 1. Five hoops were spaced at every 70 mm at tie-column's (TC's) ends to increase the shear strength, to reduce damage due to the penetration of masonry inclined cracking into the TC, and to achieve a more stable wall behavior. To reduce possible torsional displacements, orthogonal masonry walls were built. Structure 3D was tested to failure by applying cycles of quasi-static alternated lateral loads, which were linearly distributed over the height. Two cycles were applied at same displacements. A vertical stress equal to 0,49 MPa at Level 1 was maintained constant throughout the test. This stress value has been found in typical Mexican low-cost housing buildings with four and five stories. The test setup is also shown in Fig 1.

Specimen 3D was repaired and strengthened, and was retested to failure (structure 3DR). Rehabilitation works were only performed in walls at Level 1 in the loading direction. Nothing was done to slabs, bond-beams, and orthogonal walls. Wall surfaces were cleaned to improve the bond of mortar with the masonry. Cracked and crushed concrete at the ends of interior TC's was replaced with new concrete. Inclined masonry cracks were cleaned with water jet to remove the dust and crushed particles. Afterwards, cracks were filled with cement mortar and brick pieces. A welded wire mesh (150 mm x 150 mm, ϕ 3,43-mm wire) was placed on the exterior face in both North and South wall systems, and was covered with a 25-mm thick cement mortar. Nominal yield stress of the SWWM was 491 MPa. The meshes were anchored to the wall by 50-mm long nails for wood; approximately, 40 mm were driven by hand into the wall next to the wire intersection and the nail head was bent to fix the mesh. Old-fashioned metal bottle caps were left between the wall surface and the mesh to ease placement of mortar behind the mesh and to improve the mortar-masonry bond. Nail density (number of nails per area) was varied: 9 nails/m^2 were used in the North side, while 6 nails/m^2 were placed in the South face. The mesh was fixed to TC's and bond-beams (BB's) with concrete nails. The mesh terminated at the TC's edges (i.e. the mesh did not surround the TC's). The mesh was not anchored to the foundation. Prior to placement of mortar, wall surfaces were saturated. Mortar was proportioned by volume with a cement:sand ratio of 1:4. Average mortar cube strength was 10 MPa. Mortar was placed manually using masonry trowels. Structure 3DR was retested following the same load history applied to the original specimen.

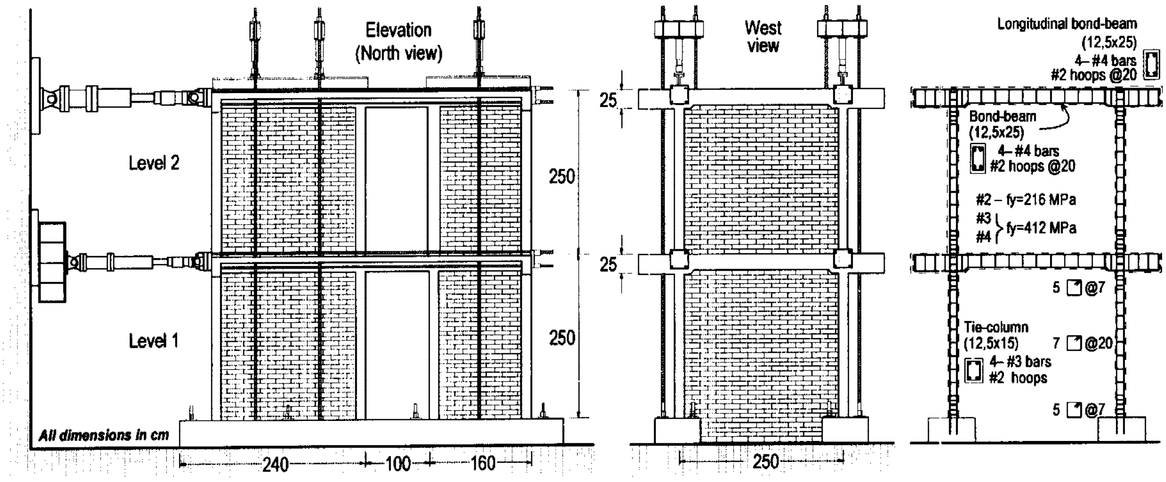
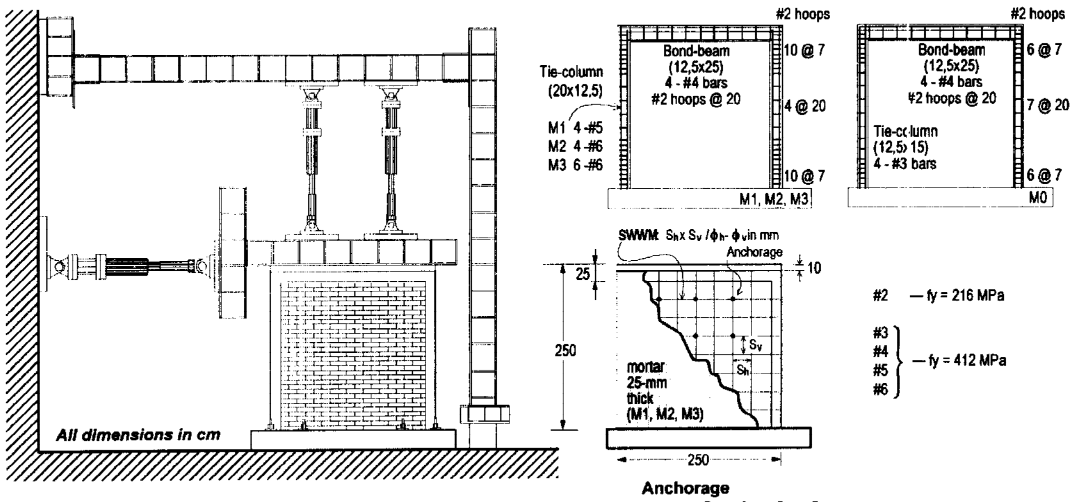


Fig. 1. Specimen for Phase A

Phase B. Strengthening of Undamaged Isolated Walls

Specimen geometry is shown in Fig 2. Structures were isolated full-scale CM walls built with hand-made clay bricks and strengthened with SWWM's. Average brick dimensions were 240 x 125 x 60 mm. Mortar used to join the bricks had a cement:sand ratio of 1:4 (by volume); the average mortar cube strength was 9,8 MPa. Average masonry strengths were 5,2 MPa in axial compression and 0,69 MPa in diagonal compression. Experimental variables were the amount of steel reinforcement provided by the mesh, and the type and spacing of anchors. Specimen M1 was reinforced with the minimum amount of horizontal steel specified in current Mexican masonry code provisions (DDF, 1995). M2 and M3 roughly had twice and thrice the minimum value, respectively (see Fig. 2). Nominal yield stress of the SWWM's was 491 MPa. Unlike 3DR, SWWM's were attached directly to both wall faces and no spacers were left between the wall and the mesh. SWWM's in M1 and M2 were anchored by 64-mm long nails for wood; nail spacing was 300 mm in one face and 450 mm in the other. Nails were driven by hand. In M3, 51-mm long Hilti anchors, driven with an impact wrench, were spaced at 450 mm in both wall faces. Reinforcing meshes on wall panels did not surround the TC's. To improve the confinement in these elements, a stretch of a mesh 150 x 150 mm / 3,43 - 3,43 mm was placed around one TC in M1 and M3, and around both TC's in M2. The confinement mesh was lapped 500 mm over the main panel mesh and was anchored only to the masonry. In all specimens, mortar cover thickness was 25 mm. Average mortar cube strength was 12,2, 7,7 and 14,1 MPa for M1, M2 and M3, respectively. The lower mortar strength in M2 was due to the different quality of the sand used during construction. The low-strength mortar had a great impact on wall behavior, as it will be discussed in the next section. Since all specimens were designed to fail in shear/diagonal tension, TC longitudinal reinforcement was calculated to attain a flexural-to-shear strength ratio of 1,5. In all specimens, slabs were 2500 x 800 x 100 mm. Results of another wall with same geometry were used for comparison (structure M0). Specimen M0 was a CM wall with no reinforcement in the panel (Aguilar *et al.*, 1996).

Specimens were tested in the rig shown in Fig 2. Loading program consisted of a load-controlled phase in which cycles were applied up to first inclined cracking in the panel. Afterwards, test was displacement-controlled through drift ratios. The latter were defined as the horizontal displacement divided by specimen's height. To assess the stability of the behavior, two cycles were done at same deformation. Similarly to Phase A, a constant vertical stress of 0,49 MPa was applied throughout. This stress was calculated using the transverse masonry area of the wall disregarding the mortar jackets.



Specimen	Wall reinforcement	$p_h = p_v$ [%]	$p_h f_y$ [MPa]	Anchorage Type	Anchorage Spacing [mm] North - South
M0	none	0	0		
M1	WWM: 150 x 150 / 3,43-3,43 [a]	0,072 [d]	0,35	64-mm nails	300 - 450
M2	WWM: 150 x 150 / 4,88-4,88 [b]	0,147 [d]	0,72	64-mm nails	300 - 450
M3	WWM: 150 x 150 / 6,35-6,35 [c]	0,211 [d]	1,03	51-mm Hilti [e]	450 - 450

[a] corresponds to standard 10-gauge wire
[b] corresponds to standard 6-gauge wire
[c] corresponds to ϕ = 0.25 inch
[d] calculated using a 175 mm wall thickness
[e] with washers 35 mm in diameter

Fig. 2. Specimens for Phase B

TEST RESULTS

Crack Patterns

Final crack patterns for all specimens are shown in Fig. 3. Damage was mainly characterized by inclined cracking over the wall panels. Cracks in walls with no steel in the panel were concentrated along the diagonals; a more uniform distribution of cracking was observed in all jacketed specimens.

As it was expected, damage in structure 3D was concentrated at Level 1. Distress was mainly by inclined cracking in the masonry panel. Minor horizontal cracks, initially due to flexure and at large drifts due to in-plane wall expansion, were observed. At drift ratios to 0,38%, masonry cracks penetrated into the TC ends. At the end of the test (drift ratio of Level 1 equal to 0,5%) the ends of interior TC's were severely damaged. Large concrete cracking and crushing, and bending of longitudinal reinforcement occurred. Few cracks were recorded at Level 2, which essentially, remained within the elastic range. Slabs exhibited flexural cracking on both top and bottom faces. Cracks were perpendicular to the loading direction, and extended along the slab width at the faces of the interior TC's. A much more uniform distribution of inclined cracking was observed in 3DR. Cracks almost covered the entire mortar cover (see Fig. 3). The interior wall faces (not jacketed) showed cracking parallel to that exhibited in 3D and repaired with brick pieces and mortar (0,21% drift ratio). Distribution of cracks in the North side was more uniform than that in the South face; it is important to recall that density of anchors was highest in the North side (9 nails/m^2). At large drift ratios of Level 1 (0,46%), mortar cover started to crush and to separate from the wall. New minor inclined and horizontal cracking ocurred in Level 2 walls due to the higher forces applied in 3DR. Crack patterns in slabs and bond-beams were not modified during the test of 3DR. At the end of the test, anchors used to fix the SWWM's were checked. Several nails had lost their anchorage or were bent and loose, especially those in the central region of the panel. Spacers used in 3DR (old-fashioned metal bottle caps) increased the flexibility of the anchor and reduced its anchorage depth, thus reducing the effective anchor strength under shear.

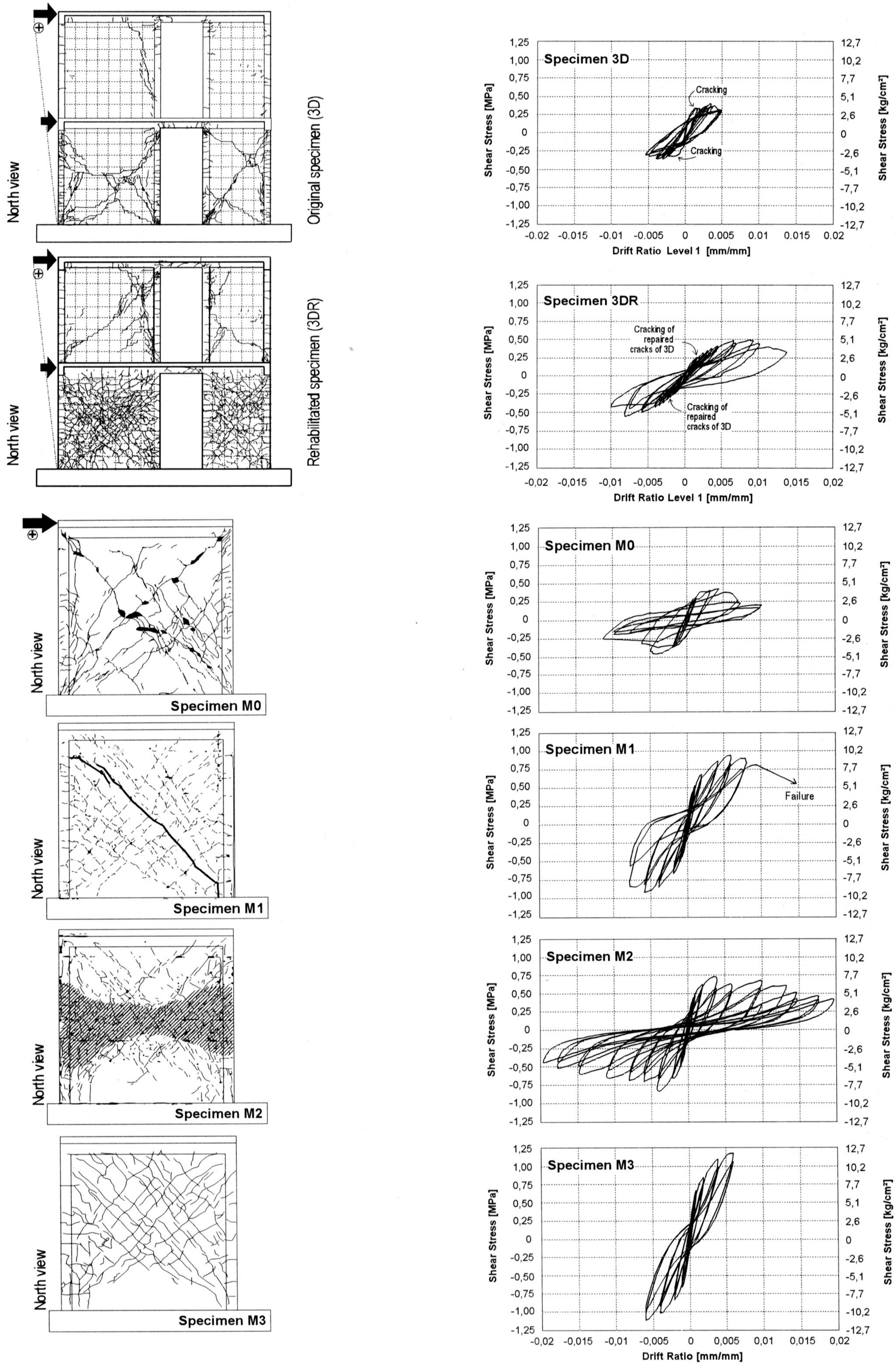


Fig. 3 Final Crack Patterns and Hysteresis Loops for Phase A and B Specimens

In Phase B, M0 showed inclined cracking concentrated along the panel diagonals. Cracks were more spreaded over the wall surface compared to the square panels in 3D due to the use of a stiff girder for lateral load distribution (see Fig. 2). At the end of the test, brick crushing and shearing of TC's were evident. Similarly to 3DR, M1 showed a very well distributed inclined cracking. The specimen failed in shear after fracture of horizontal wires along one of the wall diagonals. At failure, the TC with an extra-confinement mesh was less damaged than the opposite TC. The SWWM had slid over the TC with no extra-mesh thus accelerating the shear distress at that location. M2 failed prematurely due to mortar cracking and crushing in the central region of the wall panel. As it was mentioned, sand from a different supplier was used during construction of M2 wall jackets. Mortar strength was considerably lower (about 50%) than that measured for M1 and M3. M3 cracking was well distributed over the panel with minor horizontal cracks at wall edges due to flexure and in-plane wall expansion. Test of this structure was terminated when anchorage capacity of TC's in the foundation beam was reached. In M1 and M2, nails were used to fix the SWWM's, but no spacers were left in the mortar cover. At the end of the test all nails were found to be well-anchored to the masonry. The Hilti anchor also exhibited an excellent performance. A more uniform distribution of damage was observed in wall faces with 16 anchors/m². However, crack pattern in the face with 9/anchors/m² was acceptable.

Hysteresis Curves

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

Hysteresis loops for 3D were typical of CM structures. Cycles prior to masonry inclined cracking showed some hysteresis attributed to wall flexural cracking. The structure attained its strength at a load higher than that associated to inclined cracking. Loops were stable even at drift ratios of Level 1 equal to 0,5%. Specimen 3DR exhibited an almost linear-elastic behavior at drifts up to 0,004. Cycles at larger deformations exhibited rounding of the loading branch due to plastification of mesh horizontal wires, and energy dissipation (hysteresis) because of steel plastification, and brick and mortar cracking and crushing. Strain gauge analysis indicated that yielding of TC longitudinal steel in 3DR was reached at the base of Level 1 due to flexural deformations, whereas in 3D, plastification was due to shearing of TC's (Ruiz, 1995).

Hysteresis curves of jacketed walls were different from those obtained in control specimen M0. Although, nonlinear behavior was initiated after first inclined cracking, as in M0, loops were typical of walls whose behaviors are governed by shear. Curves were symmetric up to large drift ratios. Rounding of curves in the loading branch is attributed to yielding of panel steel. Loops showed severe strength and stiffness decay after reaching the wall strength due to damage over the panel and at TC's ends. Curves exhibited severe pinching due to shear deformations which controlled the behavior at large deformations. Pinching in M2 is also credited to the local mortar damage described before. Near failure, degradation of the behavior of M2 and M3 was accelerated by bond distress of TC rebars anchored to the foundation girder.

Response Envelopes

Behaviors of specimens can be compared through response envelopes (Fig. 4). Curves were obtained from story shear maxima at same drifts for positive cycles. Similar plots were obtained for negative cycles.

Initial Stiffness. Initial stiffness of 3DR was 2/3 of that of 3D and 6,7 times the final stiffness of 3D. Although wall jacketing did not restore the initial stiffness of the original structure, it certainly contributed to reduce the rate of stiffness decay of 3DR. Initial stiffnesses of jacketed specimens of Phase B were not affected by the amount of steel reinforcement. Similar results have been obtained in CM walls reinforced horizontally (Aguilar *et al.*, 1996).

Shear Strength. Measured shear stresses in walls are shown in Table 1. For 3D and 3D-R, values correspond to Level 1. Shear stresses at inclined cracking were larger in jacketed walls. Higher cracking stresses are explained by the superior tensile strength of the composite mortar cover - masonry wall. The maximum to cracking stress ratio was higher for 3DR and similar for specimens of Phase B. The contribution of wall jacketing to strength is evident if specimen strengths (shear stresses) are normalized by the strength of the original structure (3D) in Phase A and of the control wall (M0) in Phase B. On the average, strengths of jacketed specimens were 2,0 times those of 3D and M0. The lower value of this ratio (1,3) corresponded to 3DR, which was severely damaged prior to rehabilitation. The large increase in wall strength was due to the participation of the cement mortar and SWWM's. Wall cracking suggests the formation of compression struts in each diagonal to resist the loads. The superior tensile and compression strengths of the composite mortar-masonry over the masonry strengths allowed to resist higher forces. These struts were balanced horizontally by tensile forces resisted by the wires. The effectiveness of the wires to take the load depended on the mesh anchorage and on the behavior of mortar during cyclic excursions. To assess the participation of horizontal steel wires to wall strength, an "efficiency" factor η was derived. This factor was computed 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 reinforcement along the wall height. It has been verified that this distribution depends on the inclined crack width, on the effectiveness of the anchors used to fix the SWWM's and on the quality of mortar. Values of η calculated at wall strength are included in Table 1. Factors η were found to vary with the amount of horizontal reinforcement p_h and deformation level. The higher p_h f_y, the higher are the loads, and thus the deformations needed to mobilize the SWWM's. The latter explains the low η-factor values for M3. The small participation of SWWM to strength of M2 is explained by the failure of the mortar cover described before. In Fig. 4, the envelope of specimen MA is included. This wall had the same geometry used in Phase B and was tested by Aguilar *et al.* (1996) in a program aimed at assessing the effectiveness of horizontal steel within running bond joints for increasing the strength/wall deformation capacity of CM walls. Since the amount of horizontal steel in MA (0,071%) is roughly the same as that used in M1, their behaviors can be compared. It is clear that strengths reached with wall jacketing are much higher than those obtained with horizontal wires.

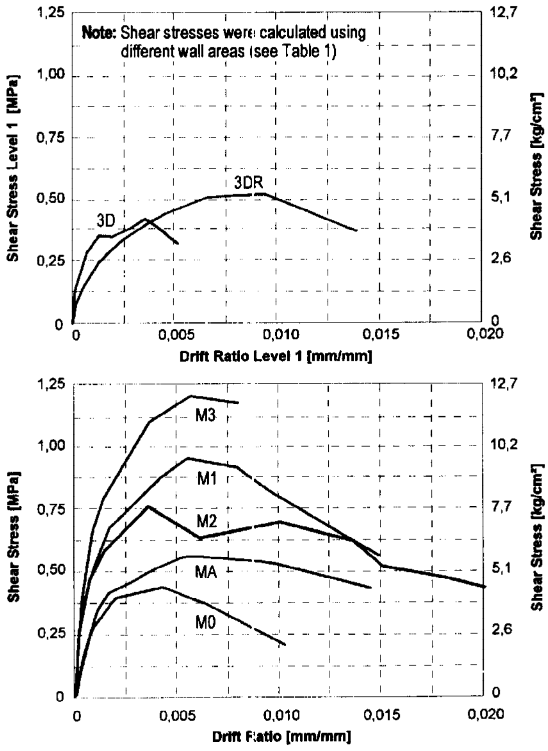


Fig. 4. Envelope curves

Table 1. Measured response characteristics

Phase	Model	p_h fy [MPa]	Shear stress[1] [MPa] Cracking	Maximum	Maximum / Cracking	Maximum / Max. control	η	Drift ratio at strength[2] [%]	Ultimate drift ratio[2] [%]
A	3D		0,35	0,40	1,1	1,0		0,36	0,47
	3DR	0,20	0,29	0,52	1,8	1,3	0,64	0,94	1,20
B	M0		0,33	0,45	1,4	1,0		0,42	0,60
	M1	0,35	0,64	0,95	1,5	2,1	0,67	0,60	0,96
	M2	0,72	0,67	0,86	1,3	1,9	0,52	0,40	0,73
	M3	1,03	0,83	1,17	1,4	2,6	0,46	0,58	1,00

Notes: [1] For 3D and M0 the total wall area in the loading direction was used; for jacketed specimens, the thickness of the mortar cover was considered.
[2] Drift ratio of Level 1

Deformation Capacity. As expected, envelope 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 due to inclined cracking, steel yielding, and brick and mortar cracking and crushing. To quantify the lateral deformation capacity of specimens, the drift ratio at strength and the ultimate drift ratio were used. The latter was defined as the drift at which 85% of the maximum load could be sustained. Lateral drift ratios for each specimen are tabulated in Table 1. It is evident that strength in jacketed specimens was reached at higher displacements that in the original and control structures (3D, M0). In Phase A, drift at strength of 3DR was almost 2,3 times that for the original structure 3D. The participation of mortar and SWWM's to deformability and strength is quite evident. Jacketing considerably increased the shear strength thus allowing the plastification of TC longitudinal steel due to flexure thus increasing the specimen deformability in flexure. In Phase B, due to mortar crushing in the panel of M2, drift at strength was comparable to that measured for control specimen M0. For jacketed walls, ultimate drift ratios were considerably higher (more than 60%) than in Specimens 3D and M0. M2 is the only exception due to the premature failure of the mortar cover reported before.

CONCLUSIONS

Based on observations during tests and on analysis of data, the following conclusions were obtained:

1. Crack patterns and failure mechanisms of specimens were governed by shear deformations.
2. Jacketed specimens with steel welded wire meshes covered with a 25-mm thick cement mortar layer showed a more uniform distribution of inclined cracking as compared to the original and control structures.
3. The initial stiffness of the repaired specimen was 2/3 of that of the original structure. The initial stiffnesses of walls jacketed with no damage were not affected by the amount of steel reinforcement.
4. Wall jacketing led to a remarkable increase in shear strength and deformation capacity. A maximum design drift ratio of jacketed CM walls is considered to be 0,007.
5. The contribution of steel welded wire meshes to strength depended on the amount of horizontal reinforcement, deformation applied, type of anchor and mortar quality.
6. Nails for wood with no spacers were found appropriate to fix meshes with wires 3,43 and 4,88 mm in diameter. Hilti anchors were effective for anchoring meshes with 6,35-mm diameter wires. A density of 9 anchors/m^2 is recommended.
7. To improve the stability of wall cyclic behavior and the confinement of tie-columns, these elements shall be surrounded by SWWM's and also covered with cement mortar.
8. Jacketed walls dissipated more energy than the original and control structures.

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