SEISMIC RETROFITTING OF LOW-RISE MASONRY AND CONCRETE WALLS USING STEEL STRIPS

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ABSTRACT: Four concrete block masonry and two reinforced concrete walls were designed to simulate low-rise nonductile walls built decades ago, before the enactment of earthquake-resistant design provisions. Two masonry walls were unreinforced and two were partially reinforced. The concrete walls had minimum reinforcement. One wall from each pair was retrofitted using a steel strip system consisting of diagonal and vertical strips that were attached using through-thickness bolts. Stiff steel angles and anchor bolts were used to connect the steel strips to the foundation and the top loading beam. All walls were tested under combined constant gravity load and incrementally increasing in-plane lateral deformation reversals. The lightly reinforced concrete walls were also repaired using only vertical strips and retested. These tests showed that the complete steel-strip system was effective in significantly increasing the in-plane strength and ductility of low-rise unreinforced and partially reinforced masonry walls, and lightly reinforced concrete walls.

INTRODUCTION

Single-story buildings are very common in North America (schools, shopping centers, hospitals, etc.), and many of them rely on shear walls to resist both vertical and lateral loads. Because of the high risk of earthquake damage to older unreinforced or lightly reinforced shear walls, and the potential for great loss of life in these important community buildings, these nonductile members have been the subject of extensive research in recent years.

Typically, these older reinforced concrete or masonry shear walls exhibit an insufficient in-plane strength and/or ductility to behave satisfactorily during earthquakes. These deficiencies can be corrected by strengthening the existing walls, using external coatings on their inside or outside face, filling existing windows or doors with reinforced concrete or masonry, or by constructing new internal or external shear walls or steel braced frames.

While the above upgrading techniques are effective, they require a great deal of preparation work, their construction may disturb the ongoing building functions, and the new structural elements may affect the architectural aesthetics of the building. Hence, an alternative method of retrofitting is worth considering. The retrofit method proposed here consists of adding diagonal and vertical strips of steel on both sides of lightly reinforced concrete and masonry walls. The diagonal steel strips that extend between the corners of the wall strengthen it while preventing diagonal tension failure and compression crushing under shear forces. The vertical strips confer a stable ductile flexural behavior to the walls. Finally, stiff steel angles and high strength anchors connecting the strips to the floors prevent sliding of the wall. The minimal increase in wall thickness due to the steel plates makes this an interesting alternative for existing walls close to mechanical equipment, such as in elevator cores.

This paper reports the results of tests on seven full-scale, low-rise masonry and concrete wall specimens. The findings experimentally validate the proposed retrofit strategy.

LITERATURE REVIEW

Numerous tests have been conducted around the world to examine the behavior of columns, beams, and slabs strengthened by the addition of steel plates (Matsuishi et al. 1980; Roeder 1984; Adams and Zimmerman 1987; Jones et al. 1988; Yoshimura et al. 1988; Harries et al. 1994; McKenna and Erki 1994; Priestley and Calvi 1996). Generally, these tests showed that this method of strengthening is an effective and convenient method to improve member strength and/or ductility. However, only a limited amount of this research is relevant to the in-plane strength of walls. Moreover, most of that previous research was conducted mainly to improve the flexural behavior of reinforced concrete elements, and few studies were carried out to improve shear behavior. Likewise, very few of these studies were done from the perspective of seismic retrofitting. In most experimental studies reported, the structural elements tested were only subjected to monotonic loading. Finally, while steel plates have been added to strengthen the walls in the elevator shaft of an existing building (Sharpe and Ugrte 1988), there is no evidence of any experimental work done on this subject.

The capability of unreinforced masonry (URM) walls to resist lateral loads is limited by the strength of both masonry units and bed joint mortar. At low axial loads, two failure modes are possible for the wall aspect-ratio considered here when earthquake forces act in the plane of the wall: (1) sliding may take place along bed joints; or (2) the wall may rock on a horizontal crack at its base, i.e., overturning around its toe. In the latter case, localized cracks and toe crushing develops at the compressed corner, but overall stability of the building is not compromised as long as deformations remain small, which is usually the case for nonslender walls. At high axial load, lateral sliding is suppressed due to an increase in bed joint friction, and diagonal cracks form within the masonry blocks (perpendicularly to the direction of principal tensile stresses) when the wall is subjected to in-plane lateral forces. Such unreinforced walls are generally incapable of withstanding severe repeated load reversals, suffering from low energy-dissipation capacity and severe strength degradation characteristics as the double-diagonal (X) shear cracking develops into extensive damage.

Reinforced masonry and concrete shear walls with a small aspect ratio can fail in diagonal tension, diagonal compression, and shear sliding (barring foundation overturning). Diagonal tension is characterized by the initiation of diagonal hairline

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cracks, when the principal tensile stresses exceed the tensile strength of concrete (or masonry) under increasing horizontal displacements. If the horizontal reinforcement is insufficient to adequately transfer the tensile stresses across the diagonal cracks, these initial diagonal cracks widen and trigger shear failure. An increase in the amount of shear reinforcement can constrain the diagonal tension cracks. This permits attainment of higher average shear stresses in the wall and the development of diagonal struts that may eventually crush under large compressive stresses. This diagonal compression failure is responsible for extreme degradation of strength and stiffness under cyclic loading. Shear failures (diagonal tension cracking and compression crushing) can be eliminated by using heavy horizontal reinforcement and relatively lighter vertical reinforcement, thus promoting flexural behavior. However, the flexural strength of such walls develops when a horizontal crack extends at their base; under loading reversal, grinding of the concrete (or masonry) occurs and a continuous weak plane forms along the crack at the bottom as a result of this crushing (even though the rest of the wall is undamaged). This triggers a sliding failure responsible for a considerable reduction in the energy dissipation and ductility of low-rise shear walls. In reinforced walls, the possibility of failure in any of these modes reduces with an increase in axial load, but, if the axial load is increased beyond a certain limit, the accompanied reduction in ductility may offset the increase in strength.

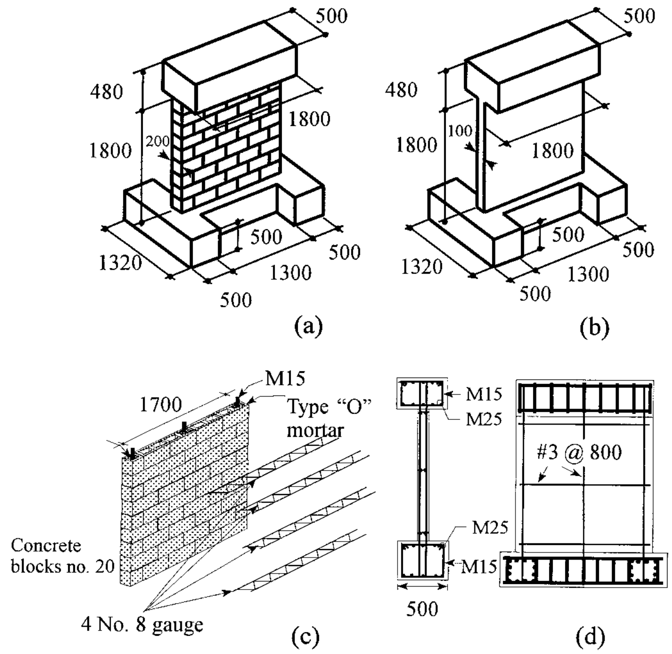

FIG. 1. Geometry of: (a) Masonry Walls; (b) Reinforced Concrete Walls; and Reinforcement Layout for (c) Wall 10; (d) Wall 11

EXPERIMENTAL APPROACH

Six large scale walls with rectangular cross sections were constructed and tested in this study. These specimens were labeled as Walls 9, 9R, 10, 10R, 11, and 11R, following the notation for eight previously tested low-rise walls (Doostdar and Saatcioglu 1998; Saatcioglu et al. 1998). The first and second pair of wall specimens were made of concrete masonry and the third pair was made of reinforced concrete. The letter R indicates ‘‘retrofitted.’’ Note that after Wall 11 had been tested, it was repaired and retested as Wall 11RP (repaired). All specimens were chosen with an aspect ratio of 1.0 (Fig. 1) to ensure that the unretrofitted walls would exhibit either shear failure or rocking failure, as described in the previous section.

The four concrete block wall specimens were prepared using standard blocks of 200 mm nominal size. Head and bed joints with an approximate thickness of 10 mm were used to provide effective masonry unit dimensions of 200 $\times$ 400 mm. Each wall consisted of a total of nine courses, providing a nominal height of 1,800 mm, as illustrated in Fig. 1. All masonry was face-shell beaded using type O mortar to represent circa-1960 construction. Details of reinforcement as well as location of the grouted cells for the partially reinforced masonry (PRM) wall are shown in Fig. 1(c). Average compressive strengths on the day of testing of 12.5 MPa and 8.1 MPa were obtained for the ungrouted and grouted masonry prisms, respectively.

Details of vertical and horizontal reinforcement for the two identical 100 mm thick reinforced concrete walls are shown in Fig. 1(d). An average concrete cylinder compressive strength of 29 MPa was obtained (on the day of testing). In all walls, Canadian 15M and American #3 rebars with yield strengths of 400 MPa and 642 MPa, respectively, were used.

Retrofit was accomplished by adding two 220 mm wide diagonal steel strips of gauge 9 thickness (3.81 mm) on each wall face, as shown in Fig. 2. Steel strips were added on both sides of the wall to prevent an eccentric stiffness and strength distribution that may cause twisting of the retrofitted walls, enhance redundancy of the retrofitted walls, and provide simultaneous retrofit against out-of-plane failures of walls (although this was not tested in this investigation). The steel strip

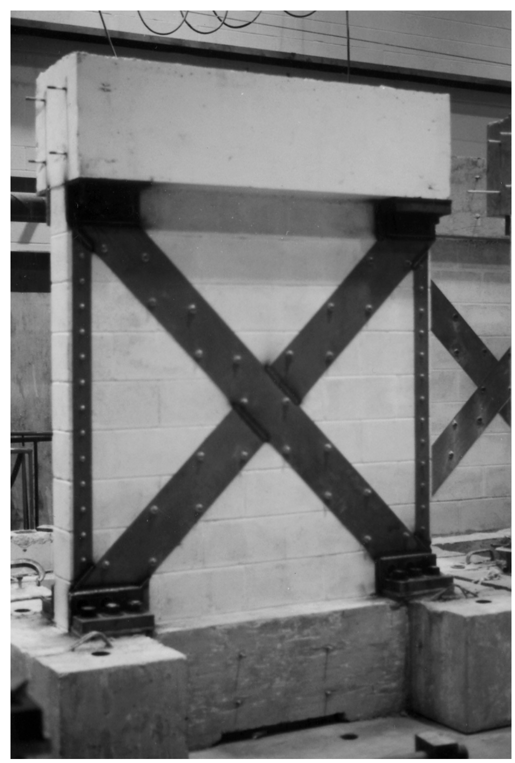

FIG. 2. Wall Retrofitted Using Steel Strips

width was chosen to ensure yielding of the gross section in tension prior to net section fracture at the bolt locations. The measured yield strength of the diagonal strips was 227 MPa. Two 80 $\times$ 3.81 mm vertical steel strips with a 248 MPa yield strength were added as boundary elements on each side of the walls, as shown in Fig. 2. Through-thickness 3/8 in. (9.5 mm) and 5/8 in. (15.9 mm) diameter A325 structural steel bolts were used to fasten the vertical and diagonal steel strips, respectively, to the walls. The spacing between these bolts was chosen to prevent elastic buckling of the strip and avoid interference with the vertical and horizontal reinforcement.

These bolts were also expected to brace the steel strips and confine the concrete or masonry in between them.

The steel strips were welded together at the center of the wall, where they meet, as well as to 300 mm long 150 × 150 × 16 mm steel angles anchored into the concrete footing and top beam using 400 mm long high-strength anchor bolts. A continuous rectangular washer plate, with three holes, was inserted between the nuts of the anchor bolts and the steel angle to prevent yielding of the angles; details of this connection are presented elsewhere (Taghdi 1998). Cement mortar bearing pads 10–20 mm thick were also used to distribute the load over the bearing area under these angles. Welding was accomplished only once the steel strips were tied to the walls and the eight steel angles and rectangular washers were installed and anchored down. The full penetration groove welds used between the steel strips were executed first.

TEST SETUP

Fig. 3 illustrates the test setup. Two vertical servo-controlled actuators applied axial compression to the specimens, and a third one was positioned horizontally and supported by a frame to apply horizontal deformation reversals. An identical axial load of 100 kN was applied to all specimens to simulate the service gravity loads that typically act on a single-story building. The setup allowed walls to be tested as cantilever elements, their footing being posttensioned to the laboratory floor, and lateral load being transferred through a stiff reinforced concrete top beam simulating a floor diaphragm.

The specimens were extensively instrumented. Strain gauges were used to measure strains in reinforcing bars and steel strips. Displacements were measured relative to the base of the wall to exclude any effect of sliding or uplift of the foundation on the laboratory strong floor. The applied vertical and horizontal loads were recorded by load cells inside the actuators. Between 10 and 36 channels of data (depending on the specimen) were recorded by two data acquisition systems. Instrumentation details of all wall specimens are presented elsewhere (Taghdi 1998).

The applied horizontal displacements under constant axial compression followed a loading history that consisted of elastic and inelastic stages. Displacements were expressed in terms of story drift, defined as top displacement divided by wall height. Specimens were deemed to have ''failed'' when their strength decay was more than 50% of the peak strength. Note that loading histories were not the same for the seven sets of walls. Each of the wall specimens were subjected to slightly different displacement histories as a result of their particular behavior and ductility.

BEHAVIOR OF WALL 9 (UNREINFORCED MASONRY WALL)

This wall behaved in a combination of rocking and sliding, as evidenced by the unsymmetric hysteresis loops of Fig. 4. The sliding developed in one direction, at an ultimate force of 64.5 kN, while the rigid-body rocking (with some small amount of sliding) developed in the other direction, at an ultimate force of −58.5 kN. The wall exhibited relatively large deformations with minor strength decay before failure. Rocking and sliding could only develop as a consequence of cracking along the bed joint. In this test, cracking did extend along the length of the wall, but the path followed by the crack was unusual. Cracking occurred neither at the base nor at the first bed joint above the base, but in the bed joint above the second course of blocks, as shown in Fig. 5. Another crack of a shorter length also appeared in the third bed joint above the base. After cracking, drift in both directions increased without any significant increase in lateral loading. Nearly symmetric

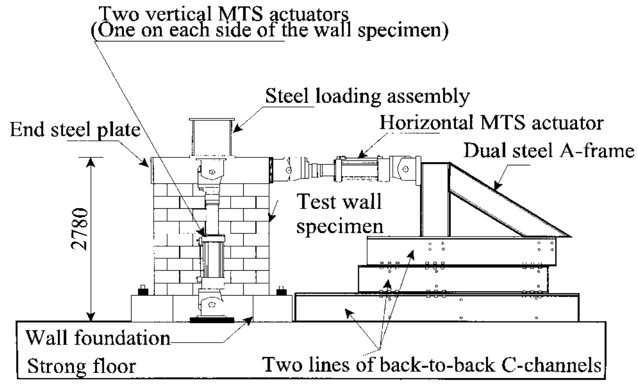

FIG. 3. Test Setup

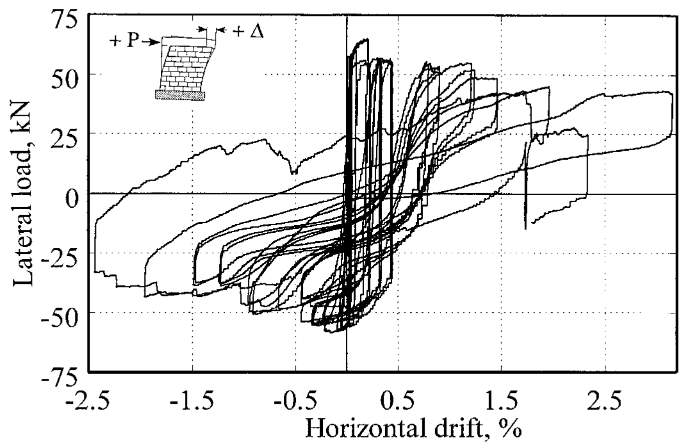

FIG. 4. Hysteretic Behavior of Wall 9

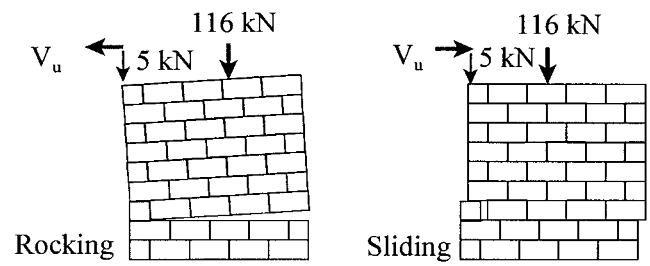

FIG. 5. Cracking and Sliding Pattern Observed in Wall 9

rocking behavior developed at drifts beyond 0.8% for reasons described elsewhere (Taghdi 1998). Despite the low strength of this wall, which indicates a certain strength deficiency, its sliding friction and rocking behavior noticeably dissipates energy.

In hindsight, even though calculations prior to testing predicted that rocking would develop at a lateral load of 46 kN, assuming cracking along the base of the wall, cracking above the base joint should not be surprising; the base joint was fully mortared in compliance with construction requirements and was thus stronger, forcing the crack to occur in the weaker joints above. However, the reason cracking occurred above the second course of masonry rather than the first is unclear at this time.

BEHAVIOR OF WALL 10 (PARTIALLY REINFORCED MASONRY WALL)

Wall 10 exhibited a symmetrical hysteretic force displacement relationship with relatively wide loops (Fig. 6). However, it suffered shear failure, with progressive crushing of masonry diagonal struts, leading to early strength degradation and relatively low energy dissipation.

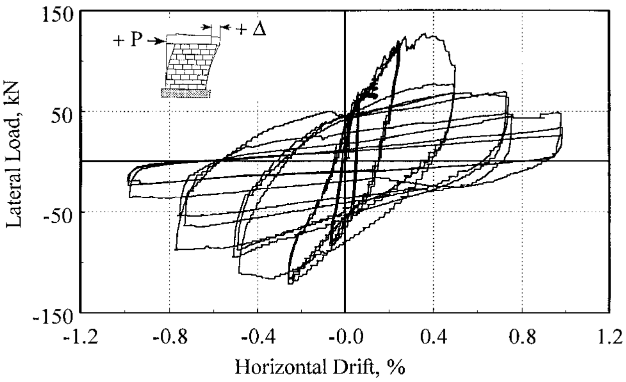

FIG. 6. Hysteretic Behavior of Wall 10

Cracks started to appear along vertical and horizontal mortar joints, from corner to corner of the wall, in a stair-step pattern at about 50% of the wall ultimate strength. The number of diagonal cracks increased with increasing load, and the cracks started to propagate through the blocks. This led to the formation of diagonal struts, as the wall developed truss behavior. At this stage of loading, the wall exhibited its maximum strength. However, because the diagonal struts could not withstand large compressive stresses, the wall rapidly suffered loss of strength and stiffness. Yielding of vertical reinforcement at the base of the wall was not observed prior to the formation of diagonal struts. The rapid loss of strength in diagonal compression struts also precluded the attainment of flexural strength. The vertical cracks that developed adjacent to the grouted masonry cells and diagonal cracks elsewhere suggest that the wall behavior changed into that of an infilled frame, with the ungrouted cells of the wall playing the role of the infill, and the grouted cells forming the columns of the frame. This behavior generated larger compressive forces at the wall corners, leading to local buckling of vertical reinforcement and crushing and spalling of masonry and mortar. Horizontal reinforcement did not appear to contribute significantly to the overall behavior of the wall.

BEHAVIOR OF WALL 11 (REINFORCED CONCRETE WALL)

Wall 11 experienced symmetrical and stable hysteretic behavior, as is shown by its force-displacement relationship in Fig. 7. These hysteresis loops also show that Wall 11 experienced rocking behavior in the later stages of testing, making it possible for this wall to maintain its strength up to 2.5% drift.

Because this wall progressively developed a rigid-body rotation behavior, with reinforcing bars controlling the wall rotation, no inelasticity was introduced in the main body of the wall panel above the base (Fig. 8). The wall ends severely crushed and the vertical rebars elastically buckled. While the strains in the end bars were not large, the rebars in the middle of the wall experienced extensive yielding and developed strain hardening at higher drifts. The wall did not experience any strength degradation until this middle reinforcement ruptured. At that stage, the wall was left to rely on pure rocking behavior to resist the lateral loads, much like the unreinforced masonry wall (Wall 9). Although the performance of this wall can be considered satisfactory in terms of ductility and energy dissipation, its lateral load resistance could be inadequate to resist earthquakes and may still require retrofit.

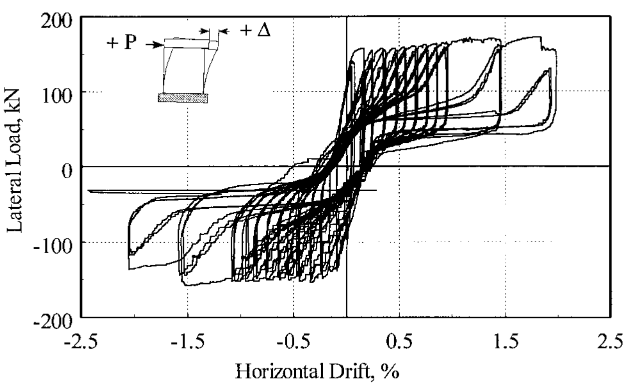

FIG. 7. Hysteretic Behavior of Wall 11

BEHAVIOR OF WALL 11RP (REPAIRED REINFORCED CONCRETE WALL)

Wall 11 was damaged along the construction joint to a degree such that there was no continuity left between the wall panel and its foundation. Damage at the ends was also localized where the reinforcement buckled. However, the rest of the wall above the construction joint did not experience any damage or even cracking. A decision was made to repair and retest this wall, as representative of a wall damaged by a severe earthquake. This was done by connecting two pairs of vertical steel strips to the ends of the damaged specimen. To illustrate potential problems that might arise when diagonal strips are neglected, the vertical strips were sized to raise the flexural strength of the wall above its shear strength. This provided an opportunity to compare the behavior of the repaired wall with vertical steel strips only with one that was retrofitted using the full steel strip system tested later.

The predicted shear strength of the virgin wall was 207 kN. Steel strips 160 $\times$ 4.8 mm ensured a flexural strength of at least 300 kN. A staggered through-thickness bolt arrangement

FIG. 8. Localized Damage in Wall 11

was selected to tie the vertical strips to the wall panel, in order to minimize strip width and ensure gross section yielding prior to net section fracture at bolt locations. The bolt spacing was chosen to avoid elastic buckling of strips. The vertical strips were welded to eight 250 mm long 154 × 154 × 16 mm steel angles, each anchored to the foundation and top beam using three 19 mm diameter 300 mm long steel wedge anchor bolts (Fig. 9).

The force-displacement relationship of Wall 11RP (Fig. 10) exhibited symmetrical and stable behavior. However, this wall experienced significant shear sliding. Slippage of the anchor bars was believed to be the main reason behind this noticeable sliding. The axial load applied to the wall did not sufficiently increase the friction resistance to prevent sliding under lateral loading. Shear cracks appeared at drifts of 1.5%, starting with two opposite diagonal cracks near the top beam. These cracks first followed a horizontal path, extending from both sides a distance approximately equal to one third of the wall length, then followed a diagonal path and intersected at about one third of the wall height (Fig. 11). This pattern indicates that the first shear cracks originated from other cracks of flexural origin. Other diagonal cracks originated from the bolt holes in the concrete. The number of diagonal cracks was limited because of the reinforcement characteristics of this wall. It is known that well distributed reinforcement promotes more evenly distributed smaller size cracks. In this repaired wall, however, only the middle rebars (even though ruptured at the wall base) remained from the original vertical reinforcement to control diagonal cracking.

The experimentally obtained lateral load capacity of the wall was 269 kN, a 52% increase in capacity compared with the virgin wall (Wall 11) and 19% higher than the shear cracking capacity calculated using ACI-318/318R-305.

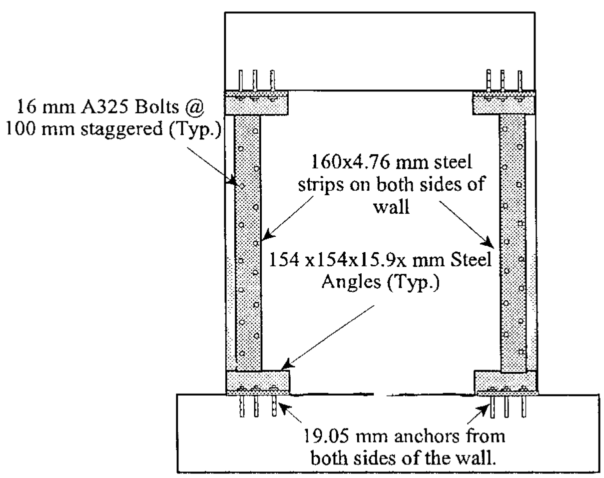

FIG. 9. Strips and Anchorage Details for Wall 11RP

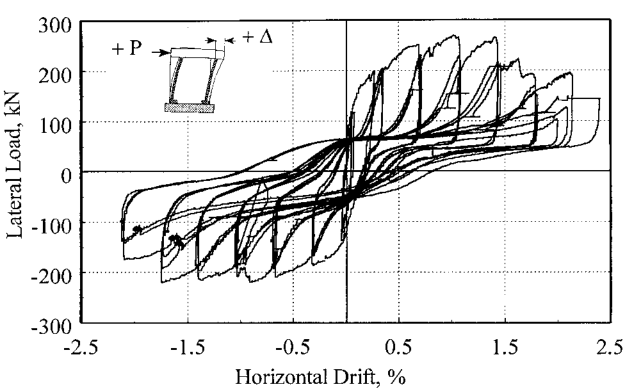

FIG. 10. Hysteretic Behavior of Wall 11RP

FIG. 11. Cracking Pattern of Wall 11RP

COMMON BEHAVIOR OF RETROFITTED WALLS

General Observations

All wall specimens retrofitted using the proposed steel strip concept exhibited superior behavior when compared with the unretrofitted wall specimens. The hysteretic behavior of Walls 9R, 10R, and 11R are shown in Figs. 12(b), 13(b), and 14(c), respectively, and compared with their respective nonretrofitted counterpart in Figs. 12(a), 13(a), and 14(a), respectively. For Wall 9R, the retrofitted URM wall, cyclic loading of progressively increasing magnitude led to some uniform cracking of the masonry, followed by yielding of the steel strips, and eventually inelastic buckling of the strips. This inelastic buckling led to the crushing of masonry. Better performance was observed in the PRM and reinforced concrete retrofitted walls, in which crushing was delayed until after the excessive yielding of vertical steel strips and rebars occurred. Fig. 15 shows Walls 9R, 10R, and 11R at 1.0% drift.

Generally, the presence of the steel strip system prevented development of the rigid body rotation observed in some of the nonretrofitted walls and allowed cracks to spread more evenly over the entire wall, which resulted in smaller crack widths. As the magnitude of the applied deformations increased, the steel strips were subjected to larger tension and compression strains. Yielding of the steel strips in tension produced permanent plastic elongations that could not be fully recovered in compression. Accumulated tensile plastic strains eventually triggered a plastic hinge midway between bolts (at one or more locations) during compression. The diagonal steel strips yielded shortly after the vertical steel strips, but, because the diagonal strips were wider and had a more favorable anchor bolt configuration, they exhibited only limited buckling.

Comparison of Hysteretic Behavior

The hysteretic relationship shown in Fig. 12(b) indicates that the retrofitted URM wall exhibits approximately symmetrical stable hysteretic behavior with significant increases in ductility, stiffness, and dissipation of energy. Fig. 12 also in-

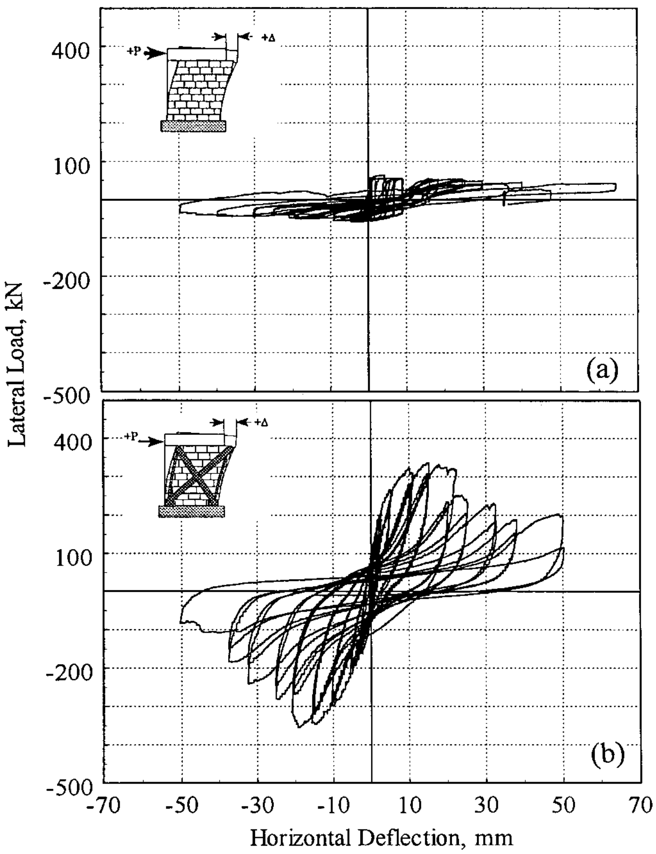

FIG. 12. Hysteretic Behavior of: (a) Wall 9; (b) Wall 9R

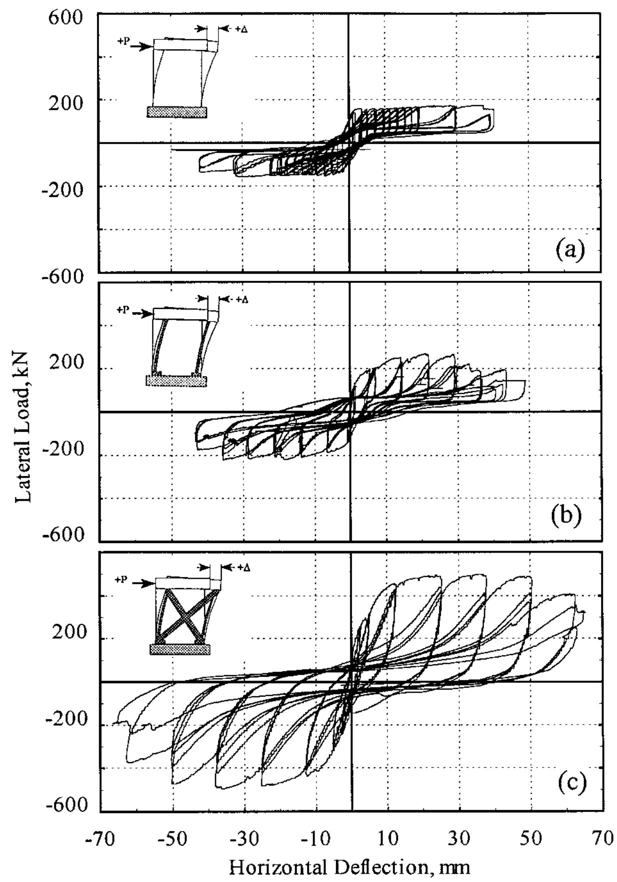

FIG. 14. Hysteretic Behavior of: (a) Wall 11; (b) Wall 11RP; (c) Wall 11R

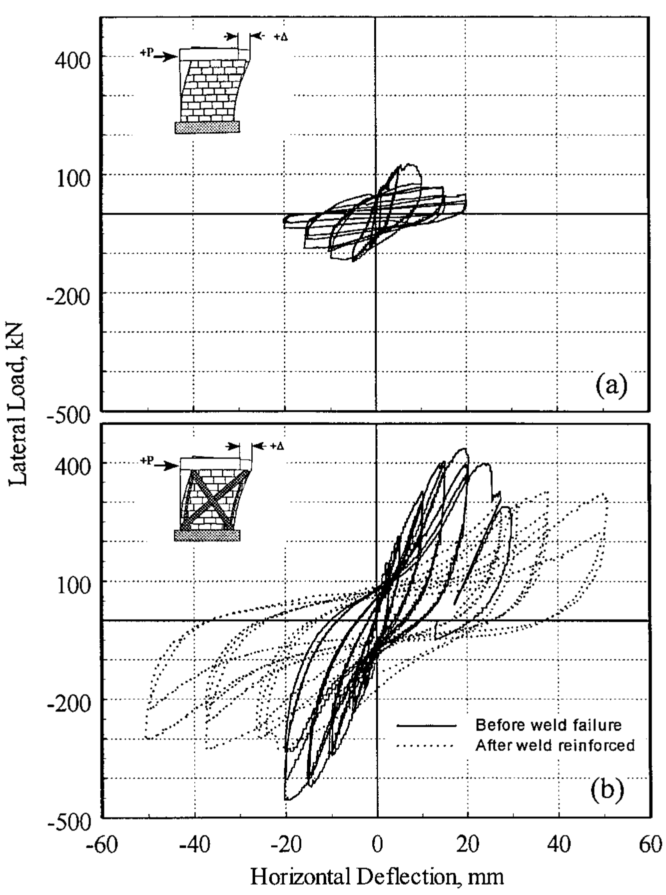

FIG. 13. Hysteretic Behavior of: (a) Wall 10; (b) Wall 10R

dicates that Wall 9R experienced a lateral load resistance 4.5 times that of Wall 9, up to drifts of 1.0%. The hysteresis loops of Wall 9R showed noticeable pinching. This pinching is attributed to bolt slippage prior to the development of composite action at low drift levels and buckling of steel strips at drifts of 0.4% and greater. Crushing of the masonry at both ends of the wall (i.e., the compression zone), contributed to the pinching of the loops. After 1.0% drift, the hysteresis loops showed a 25% strength drop with further pinching due to excessive crushing of masonry and global buckling of the vertical steel strips. The hysteretic behavior of Wall 9R resembled that of a tension-only braced steel frame where the buckled compression members contributed little to lateral resistance. In spite of this, the hysteretic behavior of Wall 9R, beyond 1.0% drift, was superior to that of Wall 9 in terms of strength, stiffness, ductility, and dissipation of energy.

The hysteretic lateral load versus top horizontal displacement relationships of the unretrofitted and retrofitted PRM walls are shown in Figs. 13(a and b). The hysteresis loops of the retrofitted PRM wall demonstrate good strength, stiffness, ductility, and overall energy dissipation compared with those of the unretrofitted PRM wall. When the hysteretic behavior of Wall 10R was compared with that of Wall 9R, it was observed that Wall 10R exhibited somewhat better lateral load resistance, stiffness, ductility, and energy dissipation. The presence of rebars and grouted cells in Wall 10R helped delay the global buckling of the steel strips. Note that some welds with poor workmanship (inadequate penetration) fractured during testing; the wall twisted about a vertical axis when weld fracture on one side led to a sudden eccentric stiffness and strength distribution. Testing resumed after repair and/or strengthening of all welds [showed by the dotted hysteretic loops in Fig. 13(b)]. Wall 10R showed a less than 7.0% drop in its lateral

(a) (b) (c)

FIG. 15. Damage at 1.0% Drift for: (a) Wall 9R; (b) Wall 10R; (c) Wall 11R

load resistance up to about 1.0% drift, whereas the lateral load resistance of the unretrofitted wall had fallen by more than 50% of the walls maximum lateral load resistance at 0.8% drift. However, once masonry crushed and spalled at the ends of Wall 10R, the shape of its hysteresis loops became similar to that of Wall 9R (the slight difference observed is because rebars were still contributing to the overall hysteretic behavior of Wall 10R at that point).

The hysteretic behavior of Wall 11R was superior to that of any other walls tested in the current investigation. The hysteresis loops, shown in Fig. 14(c), indicate high lateral load resistance, stiffness, ductility, and energy dispassion with no strength decay up to 2.0% lateral drift. The lateral load resistance dropped by 25% at a drift of 3.0%. However, the shape of these hysteresis loops still exhibited a lot of pinching, similar in shape to what is often observed in tension-only braced steel frames.

The presence of reinforcement bars in Walls 10R and 11R increased their redundancy over Wall 9R, as is described elsewhere (Taghdi et al. 2000). The anchor bolts used to attach the vertical steel strips to the wall were also helpful in enhancing their cyclic inelastic performance, as these bolts were located between the wall edges and the rebars, laterally supporting the rebars against premature buckling. The anchor bolts used in Wall 11R also delayed premature buckling at higher drift levels by confining the concrete surrounding the end rebars.

Strength

A comparison between the ultimate lateral load resistance of the unretrofitted and retrofitted walls is presented in Table 1. Wall 10R showed the highest absolute increase in lateral load resistance, whereas Wall 9R showed the lowest increase. However, these absolute increases are within 15% of each other (290.5 kN, 336 kN, and 328 kN, respectively, for Walls 9R, 10R, and 11R) and nearly the same for Walls 10R and 11R. It is believed that early crushing of the masonry at the ends of Wall 9R prevented it from developing the same increase in resistance attained by the other retrofitted walls.

Confinement

The through-thickness anchor bolts and diagonal strips provided some confinement to the enclosed masonry/concrete,

TABLE 1. Strength Increase in Retrofitted Walls

Wall label (1)	V_u (unretrofitted) (kN) (2)	V_{ur} (retrofitted) (kN) (3)	ΔV_u (increase) (kN) (4)	$\Delta V_u/V_u$ (increase) (%) (5)	V_d[a] (kN) (6)
9, 9R	64.5	355	290.5	450	243
10, 10R	120	456	336	280	243
11, 11R	171	499	328	192	243

[a]Horizontal component of yield strength of diagonal steel strips in tension.

which in turn improved the behavior of the masonry/concrete in the compression zone. However, when the vertical and diagonal steel strips buckled, their compressive strength progressively decreased, transferring the burden of compressive resistance on the masonry/concrete struts. This redistribution accelerated the crushing of masonry in the case of Wall 9R (URM) and to a lesser extent for the other retrofitted walls. Thus, the benefit of confinement was rapidly lost in Wall 9R, because of the low masonry strength. However, it was beneficial in Wall 11R, which could reach drifts of 2.5% without significant strength degradation.

Behavior of Steel Strips

All vertical steel strips behaved in a similar manner, with local buckling typically developing at the top and bottom ends [Fig. 16(a)]. Pinching in the hysteretic behavior of the retrofitted specimens increased with the magnitude of this local buckling. The vertical strips of Wall 9R ultimately suffered global buckling after crushing of masonry at the edges of the wall, as shown in Fig. 16(b). This translated into severe strength decay starting as early as from 0.8% drift. The vertical strips in Walls 10R and 11R, better restrained by the reinforced cells and concrete, respectively, were able to sustain larger strains. Ultimately, the vertical strips at the bases of those two walls fractured due to low-cycle fatigue under excessive plastic strains at the midlength of the local buckles. It is believed that, as a complementary procedure to retrofitting Wall 9R, grouting of the end cells would help delay their crushing, prevent global buckling of the vertical strips, and attain higher drifts without any strength deterioration. However, this remains to be verified experimentally.

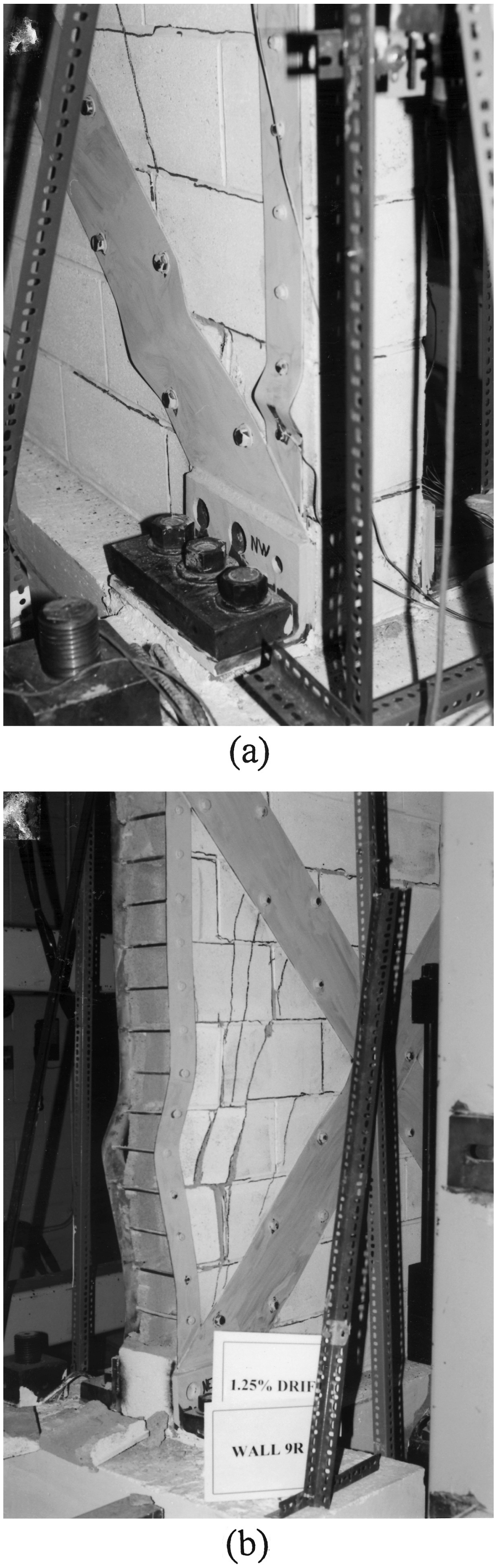

FIG. 16. Buckling Modes of Steel Strips: (a) Local Buckling; (b) Global Buckling

ENERGY DISSIPATION

The hysteretic energy dissipated during the first cycle at each drift level is presented in Fig. 17 as a function of lateral drift, for all the walls. Walls 11R and 10R dissipated nearly the same amount of energy per cycle and the largest amount

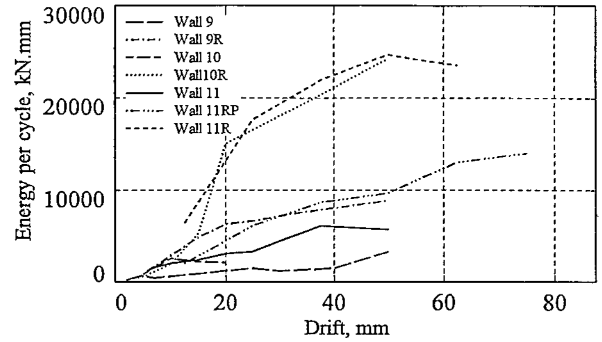

FIG. 17. Hysteretic Energy Dissipated during First Cycle at Each Drift Level

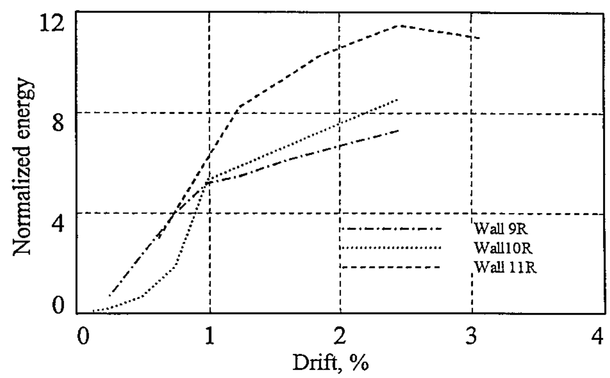

FIG. 18. Hysteretic Energy Dissipated during First Cycle at Each Drift Level, Normalized by $R_y\Delta_y$

of all walls tested. Wall 10 dissipated the smallest amount of energy overall, but testing was stopped when its strength dropped to 50% of the maximum capacity reached, which occurred at relatively lower drifts. At equal drifts, Wall 9 actually dissipated the lowest amount of hysteretic energy. To account for the fact that walls had different amounts of reinforcement, steel strip sizes, and strengths, normalized hysteretic energies were calculated by dividing the previously calculated hysteretic energy of each wall by the product of its yield strength and yield displacement. Fig. 18 presents these results as a function of drift for the three retrofitted walls. In that perspective, Wall 11R showed a superior normalized energy dissipation capability. Walls 9R and 10R had similar results at drifts above 1%, with Wall 9R exhibiting a superior normalized hysteretic energy at lower drifts.

Note that, in the retrofitted walls, truss action dominated behavior and hysteretic energy was dissipated primarily by yielding of the steel strips. In spite of pinching in the hysteresis curves, the steel strip system provided an effective energy dissipation mechanism.

CONCLUSIONS

Experiments conducted in this study show that the steel strip system, proposed to retrofit low-rise masonry and concrete walls, is effective in significantly increasing their in-plane strength, ductility, and energy dissipation capacity. For the particular specimens considered, addition of steel strips increased the lateral load resistance of each wall by approximately 300 kN. The details and connections used to ensure continuity between the steel strip system and the foundation and top beam also enhanced the sliding friction resistance.

Note that, although no out-of-plane tests were conducted within the scope of this work, the writers believe it is preferable to use the proposed strip system on both sides of the wall, to provide greater out-of-plane strength and minimize out-of-plane displacements. Finally, it is shown that the anchor bolts along the vertical strips can be placed to provide lateral supports to the end bars of the existing reinforced concrete/masonry walls, helping to eliminate their premature buckling. Also note that existing walls retrofitted using only vertical steel strips are not as effective, as their ultimate strength can still be limited by their less-ductile shear failure.

Tests of the nonretrofitted walls, beyond providing a basis for comparison of the retrofitted walls, also made some valuable observations possible. In particular, the stable in-plane rocking mode of failure, experimentally observed in unreinforced brick masonry walls by other researchers, was shown to develop in the concrete block walls tested here. The tests also suggested that partially reinforced masonry walls tend to behave in a manner similar to infill frames. Finally, buckling of the end reinforcing bars at rather low drifts in the existing concrete wall provided further evidence to support building code requirements for boundary members.

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