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SEISMIC RESPONSE OF END-CONFINED REINFORCED CONCRETE BLOCK SHEAR WALLS

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ABSTRACT

A potential drawback to reinforced masonry shear wall construction is that common practice and practical limitations result in flexural reinforcement placement as a single layer along the centre of the wall. A reinforcing pattern of this type is susceptible to stability problems under in-plane cyclic loading especially at the wall toes. Enhancing the stability of the compression toe at high deflection levels has been carried out by adding boundary elements to linear walls. Adding boundary elements to linear walls (end-confined walls) resulted in significantly improving the stability of the compression zone, delaying bar buckling and facilitated achieving high levels of deformation and ductility prior to failure. The data presented in this paper is a part of an ongoing experimental and analytical investigation of the response of reinforced masonry shear walls having variable end configuration and subjected to different axial compressive stress. This paper presents the experimental results of three end-confined reinforced masonry shear walls subjected to different axial compressive stress. The walls were tested under reversed lateral cyclic displacement simulating earthquake excitation and were subjected to axial stresses of 3%, 6%, and 9% of the experimentally obtained masonry compressive strength. Details of the test walls as well as the test setup, instrumentation and material properties are also presented. All walls demonstrated high levels of ductility at the three different axial load levels (low, typical and high) with highest and lowest ductility levels corresponding to the walls subjected to the lowest and highest axial compressive stress, respectively. Results showed that significantly higher ductility levels than currently perceived can be easily achieved through the addition of boundary elements.

KEYWORDS: boundary elements, ductility, reinforced masonry, seismic performance.

INTRODUCTION

Major losses during recent earthquakes have led to the adoption of more stringent seismic design requirements in North America. This is particularly true for low and moderately active seismic regions and has especially affected the design of masonry buildings which are perceived to have less ductility and be more vulnerable to seismic loading compared to their reinforced concrete counterparts, especially in Canada.

A widely held belief is that masonry cannot provide high ductility. However, the results of recent experimental research at McMaster University [1-2], and in other parts of the world [3], showed that this is not true. These experimental results demonstrated that reinforced masonry shear walls failing in flexure can achieve high ductility and slow strength degradation under cyclic loading. The lateral load capacity of reinforced masonry shear walls was shown to be maintained for drift levels beyond those corresponding to maximum load, with almost no degradation of lateral load capacity even after toe crushing and the development of end block faceshell spalling. It was also shown that only after splitting of the outermost grout column and buckling of the end reinforcing bars degradation of strength becomes significant [1]. In this regard, a masonry shear wall having a single line of vertical reinforcement can almost have no confinement at the compression zone. Such masonry shear walls may be susceptible to buckling of the vertical bars in compression and out-of-plane displacement of the wall during reversed cyclic loading [4]. Hence, confinement of the wall ends is a strategy that is expected to delay splitting of the grout column and buckling of the end bars and, therefore, should increase the displacement ductility by delaying strength degradation.

The behaviour of three fully grouted reinforced masonry shear walls tested under different axial load levels is presented in this paper. The aim of this study is to document and evaluate the effects of different axial load levels on the lateral response, ductility capabilities and inelastic deformation of flexuraly designed end-confined reinforced masonry shear wall.

EXPERIMENTAL PROGRAM

Although a relatively large amount of experimental data are available for linear reinforced masonry walls, little data is available on reinforced masonry walls with boundary elements and closed ties. It is well established that varying the axial load affects the moment capacity and the displacement ductility of shear walls. Since the effectiveness of a boundary element is dependent on the size of the compression zone, the impact of axial load on the effectiveness of boundary elements on ductility is also important. The test walls were designed to investigate the post-peak response of end-confined masonry shear walls under varying the axial compressive stress. All walls were subjected to fully-reversed displacement-controlled quasi-static cyclic loading and were loaded up to 50% degradation of strength in order to obtain enough information on the post-peak behaviour.

Type S mortar, with an average flow of 125% was mixed by weight with proportions of Portland cement: Lime: Dry sand: Water = 1.0: 0.2: 3.5: 0.9. Fine grout mixed in the laboratory was used for grouting the walls. The average cylinder compressive strength of the grout was 21.3 MPa (*c.o.v. = 15.8%)*. Grout filled 4-block high prisms were constructed in running bond to determine the wall properties. The average compressive strength of the grouted masonry prisms, f'_m, was 15.1 MPa (*c.o.v. = 14.3%)*. Tensile tests conducted on the vertical reinforcement gave an average yield strength of 496 MPa (*c.o.v* = 2.3%). The concrete used in the wall foundation had an average compressive strength of 26 MPa *(c.o.v. = 2.9%)*. Concrete mixed in the laboratory, having an average compressive strength of 35 MPa (*c.o.v. = 8.4%),* was used in the three slabs representing storey levels.

For a typical 5 storey masonry building, wall lengths between 2 m to 8 m long result in an aspect ratio of at least 1.5 (storey height is about 2.4 m), and axial compressive stress can vary from about 1 MPa to 2 MPa (0.2 to 0.4 MPa per floor). The design of the walls in this part of the experimental program was based on the stated range of aspect ratios and axial compressive stress. A typical compressive stress level of 0.3 MPa per storey was selected and the low and high axial stresses are selected as 50% and 150% that of the typical load to cover a wider range.

WALL DETAILS AND CONSTRUCTION

Laboratory testing of full-scale masonry walls can be impractical due to space limitations, construction and testing constraints, and time and financial restrictions. Even with the 12 m head room and the strong floor in McMaster's Applied Dynamics Laboratory, large full-scale structures cannot be built and tested. An alternative solution was to model full-scale elements using half-scale masonry units. This approach was shown to closely simulate full-scale construction [2]. A reasonable specimen height to be tested in the laboratory was estimated to be about 4.0 m. Therefore, when using half-scale blocks, assuming that the floor height to be about 1.2 m (2.4 m in full-scale construction) the corresponding number of stories to model is 3. Based on the previous values, the wall dimensions were selected to be to 1.80 m long 3 3.99 m high (3.6 m3 8.0 m in full scale construction) which results in an aspect ratio of 2.2.

The construction of the test walls started with pouring of the reinforced concrete base. This was followed by construction of a wall up to a storey height which was followed by grouting the wall solid before construction of the reinforced concrete slab representing the storey floor. An experienced mason constructed all the walls in running bond with the half-scale hollow concrete masonry units using face shell mortar bedding and 5 mm (half-scale) mortar joints. The length of the walls consisted of nine and a half (half-scale) concrete blocks, and the height of the walls consisted of 39 courses (13 courses per storey) and 3 reinforced concrete slabs (each of 100 mm thick). All specimens were fully grouted and the vertical and horizontal reinforcement were uniformly distributed over the wall. The three concrete slabs were reinforced in two orthogonal directions and were cast at wall heights representing the floor of each storey. The slabs extended the whole length of the wall and extended laterally 150 mm from each side.

All three test walls had the same cross section dimensions and reinforcement scheme as shown in Figure 1. The walls comprised of a 90 mm thick web and a four-celled boundary element at each end measuring 185 mm × 185 mm. The web of the wall was reinforced with 3-10M (100 mm^2) bars spaced every fourth cell and 4-10M bars in every cell in both boundary elements (ρ_v = 0.56%). Horizontal reinforcement, consisting of D4 wires (25.4 mm^2), were placed along the notch located in the webs of the blocks and were tied to the outermost vertical reinforcement in the boundary elements. Reinforcement was placed in every course for the first floor (ρ_h = 0.30%) and every other course in the second and third floors (ρ_h = 0.15%). This scheme was selected in accordance to the CSA S304.1-04 specifications for shear reinforcement in seismic areas and the special consideration needed within the expected plastic hinge region.

The reinforcement ratios, number of bars, and level of applied axial stress for the test walls are listed in Table 1. All walls have the same reinforcement scheme and overall dimensions. Axial loads of 0.45, 0.89 and 1.34 MPa were applied to Walls 1, 2, and 3, respectively, and this was the only test parameter.

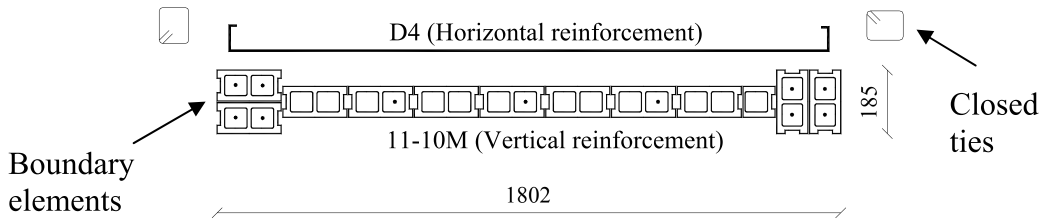

Figure 1: Details of Cross-section and Reinforcement in the Test Walls

Table 1: Summary of Wall Details

Specimen	Dimensions	Vertical Reinforcement		Horizontal Reinforcement	Axial Stress
	Length x Height (mm)	Bars	ρ_v (%)	D4 @ (ρ_h (%))	MPa
Wall 1	1,802 x 3,990	11 x 10M	0.55%	95 mm 1st floor (0.3%) 190 mm 2nd and 3rd floors (0.15%)	0.45
Wall 2					0.89
Wall 3					1.34

TEST SETUP

The test rig was designed to test shear walls up to 3.0 m long under reversed cyclic loading. As shown in Figure 2, the rig consisted of a 4,200 mm long × 1,100 mm wide × 600 mm deep reusable concrete floor slab that was prestressed to the structural floor with the aid of ten, 63 mm diameter, post-tensioned steel bolts spaced at 920 mm in both the longitudinal and transversal directions. Sixteen 25.4 mm diameter steel prestressing bars were anchored in the reusable floor slab and, after positioning the test wall, were post-tensioned to clamp the wall base to the reusable floor slab in order to prevent its rotation during wall testing. These prestressing bars were spaced at 400 mm in the longitudinal direction and at 320 mm in the transverse direction.

Axial load was applied to the top of the wall by means of four 13 mm diameter high strength prestressing rods that were bolted to a steel beam (placed orthogonally to the direction of the top loading beam) and attached to the reusable concrete slab. Each pair of bars pivoted on a roller oriented along the length of the wall. Load was applied by a hydraulic jack on one side of each pair of the prestressing bars and was distributed along the wall length through the top steel loading beam. The lateral cyclic load was applied using a hydraulic actuator positioned to coincide with the top of the wall in order to create a zero moment condition at the top of the wall. The actuator was attached to the stiff steel loading beam at the top of the walls to which the vertical reinforcement of the wall was welded. Steel dowels (10M) were inserted, during grouting of the second half of the 3rd storey, in all the cells not containing vertical reinforcement. These dowels extended into the 2 top masonry courses and to a height of 200 mm above the top course and were then welded to the top beam in the cells not containing vertical reinforcement. This arrangement was selected to uniformly transmit the lateral load along the whole length of the wall instead of as a point load at the top corner of the wall.

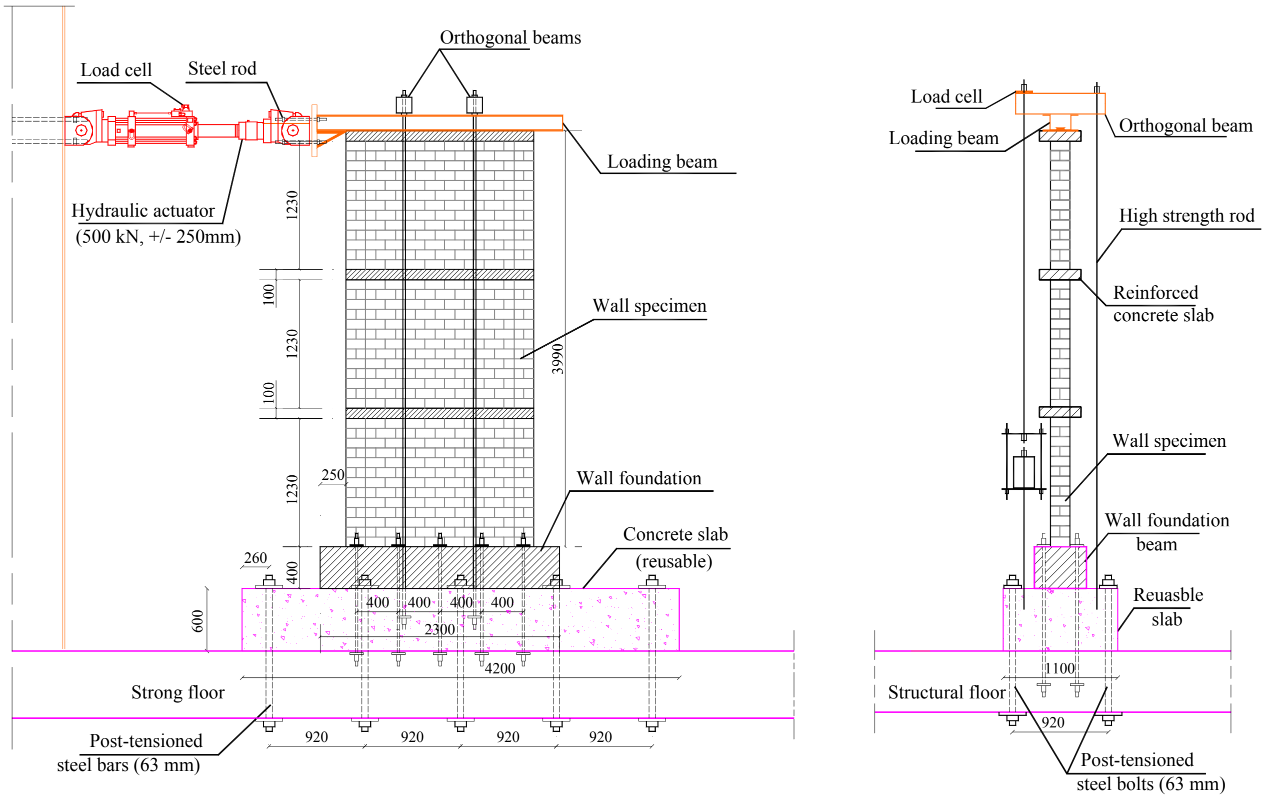

Figure 2: Test Setup

The walls were braced against out-of-plane displacement using two hollow steel link members pinned to a steel frame and connected to each reinforced concrete slab representing the storey floor. Box sections were attached to the out-of-plane bracing frame and to the reinforced concrete slab at each storey with 25 mm high strength threaded rods to create pinned connections. The two link members at each storey were designed to offer minimal resistance to the in-plane displacement of the loading beam and to prevent out-of-plane movement of the wall at all stories during the test.

INSTRUMENTATION AND TEST PROCEDURE

Thirty-six potentiometers were used to monitor lateral deflections, vertical deformations, diagonal deformations, slip along the base, and wall uplift. The vertical displacements of the walls were monitored by the potentiometers installed vertically along the wall ends. Each of these potentiometers measured the vertical movement of the storey relative to the concrete slab beneath it and was used to calculate average curvature over that segment of the wall height. The lateral displacements of the wall at different heights were measured by seven horizontally positioned potentiometers. In addition, electrical strain gauges were epoxied to the reinforcing steel bars prior to wall construction. The gauges were located within the most highly stressed region to monitor initial yielding, extent of yielding over the wall height, and penetration of yielding inside the foundation. Stain gauges were located within the wall foundation, at the interface between the wall and foundation as well as at ⅓ and ⅔ of the wall height.

A displacement controlled loading procedure was used until yielding of the outermost bar occurred based on the reading of the strain gauges attached to the bar at the interface between the wall and the foundation. The initial displacement cycles were based on reaching 0.2, 0.4, 0.6, 0.8 and 1.0 times the theoretical yield resistance of the wall, which was within 5% of the measured resistance based on the readings of the strain gauges. Then, for subsequent displacement cycles, the walls were tested using increments equal to multiples of the displacement recorded at the onset of yielding of the reinforcement, Δ_y. Based on the first test (Wall 2) in which fracture of the vertical reinforcement occurred, the number of cycles before yield was reduced and the step size in cycles after yield were increased in an effort to reduce fatigue of the bars as done in Walls 1 and 3.

EXPERIMENTAL RESULTS: WALL 1 – LOW AXIAL LOAD

The hysteresis loop for Wall 1 is shown in Figure 3 and shows a generally symmetric response in both loading directions until very high displacements. Initial loading of the wall up to the yield displacement resulted in an almost linear elastic response characterized by thin hysteresis loops generating low energy dissipation. Whereas, under higher displacements, reduction in the loading stiffness and increased energy dissipation, can be observed from the bigger loops. Yielding of the outermost reinforcement bars was recorded at 103 kN and 9.8 mm in the push direction (+)ve and at 103 kN and 10.3 mm in the pull direction (-)ve. Due to the slight time lag of when a measurement is scanned and when a visual readout appears a yield displacement of 10.5 mm was recorded during the test and used for further cycles despite the fact that further analysis of the data seems to indicate a slightly lower value of 10.1 mm.

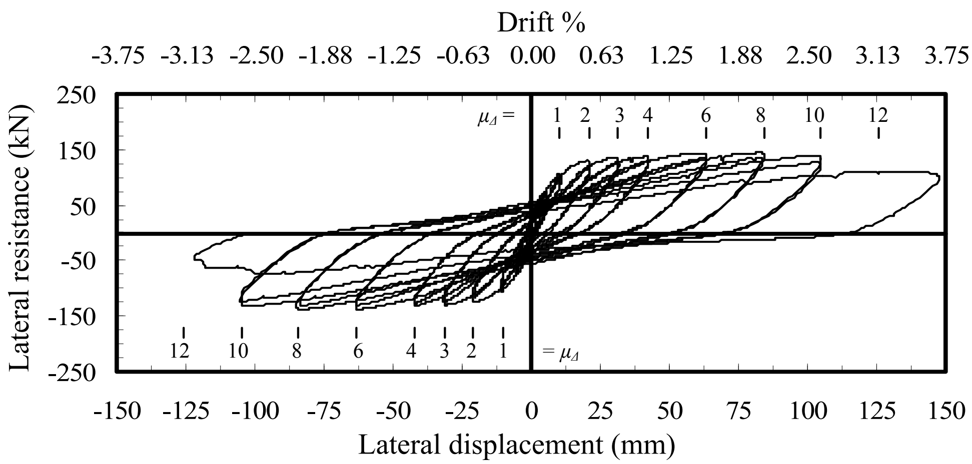

Figure 3: Hysteresis Loops for Wall 1

The first crack in the wall was observed at 75% the yield load between the fourth and fifth courses as well as between the seventh and eighth courses along the bed joints. Diagonal cracking began to appear at 21 mm displacement (23Δ_y). Few horizontal and diagonal cracks in the web of the wall at the second floor were observed at 42 mm (43Δ_y) displacements. The wall reached its maximum load of 146 kN in push and 140 kN in pull at 84 mm top displacement (83Δ_y). At 105 mm top displacement (103Δ_y) there was a minimal drop in capacity (4.2% in push and 4.7% in pull) as the first vertical compression cracks appeared in the toes. Despite this, the grouted core remained intact. Displacing the wall to a target 147 mm (143Δ_y) top displacement in the push direction, fracture of one of the outermost reinforcement bars under tension occurred at approximately 110 mm (2.8% drift). Once the wall reached the target displacement its capacity was 106 kN, a drop of 27% from the ultimate capacity due to the bar fracture. It was also observed in the compression toe that total spalling of the block had occurred and bugling of the compression reinforcement had begun between the foundation and the first tie at 80 mm although the grouted core appeared to be relatively intact thus far. Upon cycling the wall in the pull direction crushing of the compression toe began at 98.5 mm followed by fracture of a second bar at 116.5 mm.

EXPERIMENTAL RESULTS: WALL 2 – MODERATE AXIAL LOAD

The hysteresis loops for Wall 3, shown in Figure 4, indicate a symmetric response for loading in both directions. Yielding of the outermost reinforcement was recorded at 110 kN and 9.0 mm displacement for loading in the push direction (+)ve, and at 106 kN and 9.4 mm displacement for loading in the pull direction (-)ve. The increased displacement for each cycle was based on a yield displacement of 9.2 mm.

As the hysteresis loops show, Wall 2 reached a maximum capacity equal to about 152 kN at 36.8 mm ($43\Delta_y$) top lateral displacement. The wall did not loose any significant amount of lateral capacity until 92 mm ($103\Delta_y$) top lateral displacement, but during the second loading cycle at this displacement, the wall lost about 15% of its maximum lateral capacity. Unfortunately, as can be seen, this is likely due to having accidentally loaded the wall to a top displacement of 124 mm instead of 92 mm at the beginning of the cycle. The 124 mm displacement (more than $133\Delta_y$) was imposed without loss of capacity but did cause additional yielding of the tension bars which affected subsequent cycles of loading. At the 92 mm displacement level, crumbling of the unconfined grout column (outside of the ties) occurred over the lower course and the vertical reinforcement buckled between the base of the wall and the first confining tie located at 80 mm above the base. A significant loss in strength occurred during loading to a target displacement of 101.2 mm ($113\Delta_y$) due to fracture of the vertical reinforcement which is likely due to low cycled fatigue. The data from the accidental loading to a top displacement of 124 mm indicates that the wall would have produced even higher ductility capabilities than shown if fewer cycles of loading had been applied at lower displacement levels to reduce the fatigue effect.

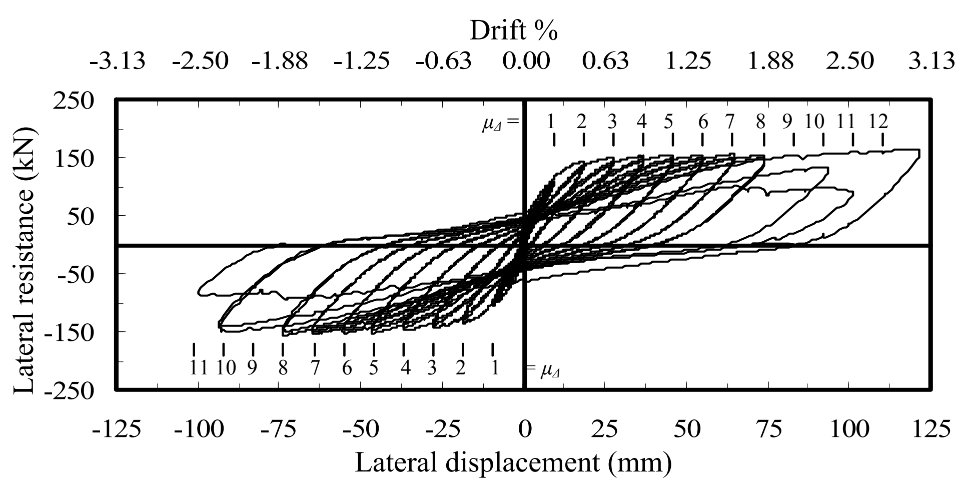

Figure 4: Hysteresis Loops for Wall 2

EXPERIMENTAL RESULTS: WALL 3 – HIGH AXIAL LOAD

The hysteresis loops for Wall 3, are shown in Figure 5. After cycling the wall at two-thirds the expected yield load, a problem occurred in the software controlling the hydraulic jack and resulted in an instantaneous push for the wall. The recorded values for lateral wall displacement and wall resistance before turning off the hydraulic system was 92.2 mm of displacement (2.3% drift) and 195 kN. Spalling of the face shells in the boundary element subjected to compression occurred due to this accidental loading but, fortunately, because of the confinement offered to the compression toe, the grouted core remained intact. Large amounts of horizontal and diagonal shear cracks were observed in the wall localized to the first floor.

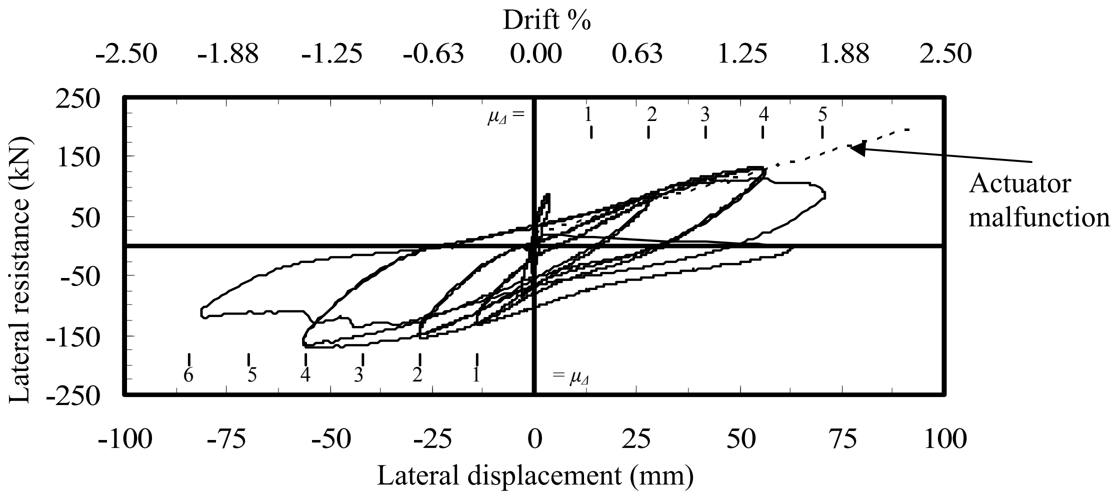

Figure 5: Hysteresis Loops for Wall 3

Testing of the wall was resumed by cycling the wall to its theoretical yield load and then based on multiples of the recorded yield displacement. Because the strain gauges used to measure the reinforcement were no longer reliable after the accidental loading, the wall was fist loaded to 133 kN in the pull direction (the theoretical load at which yielding should occur). The recorded yield displacement during loading in the pull direction was 14.0 mm. It is expected that the exact yield displacement, in case no accidental error has occurred, would have been lower as the wall lost some stiffness due to the damage of the face shells at one end and the yielding of bars at the other end. However, it was decided to proceed with cycling the wall based on 14 mm intervals since there was no other way to identify the exact yield displacement.

At 70 mm top displacement ($5\Delta_y$) in the push direction, crushing of the confined grouted core occurred and damage quickly spread to the web of the wall and the wall resistance dropped to about 107 kN. Buckling of the reinforcement occurred and was localized at the second course between the first and second ties located at 80 mm and 175 mm above the foundation. It was decided that at this point to only load the wall monotonically in the pull direction since its compression toe remained completely intact. During loading to a target 84 mm top displacement ($6\Delta_y$), the vertical reinforcement in the boundary element began to fracture on the tension side at 34.7 mm, 44.2 mm and 53.3 mm respectively. Despite losing three of the four bars in the boundary element on the tension side a load of 119.8 kN was reached at $6\Delta_y$. However, due to the loss of nearly half the tension carrying bars the test was terminated.

EFFECT OF THE AXIAL COMPRESSIVE STRESS

The load-displacement relationships for the test walls are presented in Figure 6 (a). The axial load had a minimal effect on the moment capacity of the walls but an increase in the attained displacements (drift) prior to significant strength degradation was observed with decreased axial load

The increase of the applied axial compressive stress from 3% to 9% of the masonry compressive strength resulted in a decrease of drift capacity at maximum load from about 2% to 1.5% and from about 3.6% to 2% at 30% strength degradation (70% ultimate load on the descending branch) as shown in Figure 6 (b).

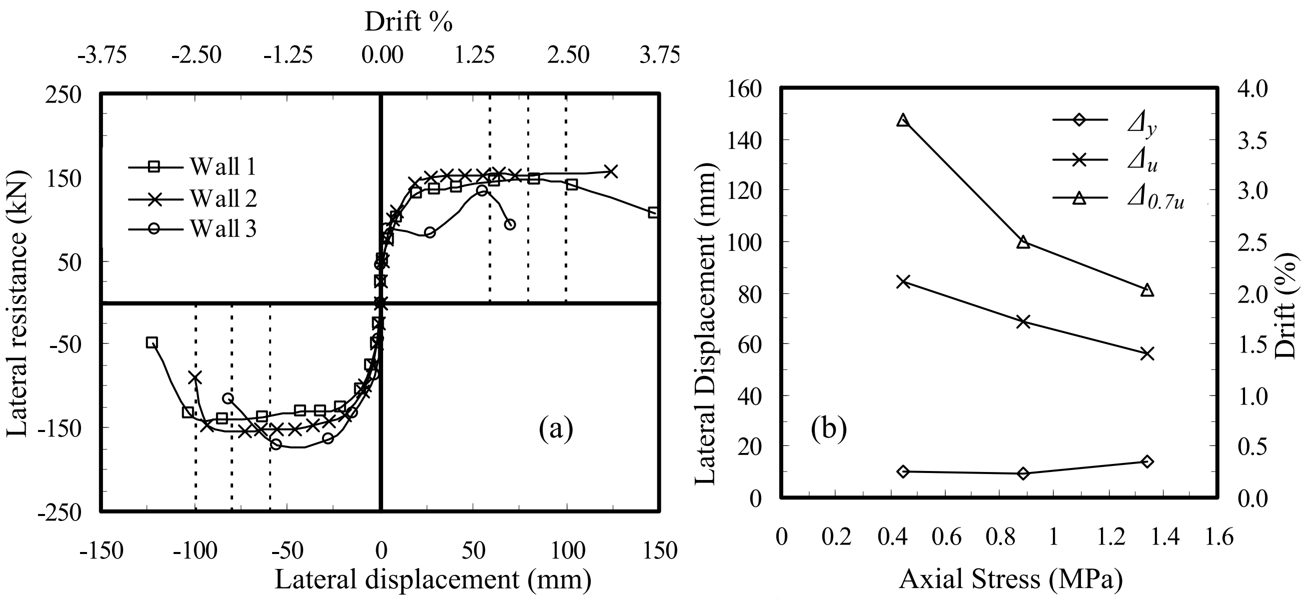

Figure 6: Load Displacement for Test Walls

The test results demonstrate that masonry shear wall can undergo significant plastic deformations without a significant loss of strength ($103\Delta_y$ for Walls 1 and 2) with the addition of confinement in the toes through boundary elements. Wall 3 was able to survive the instantaneous accidental loading and the damaged wall reached high drift levels and generated large hysteresis loops with significant plastic deformation and energy dissipation before failure.

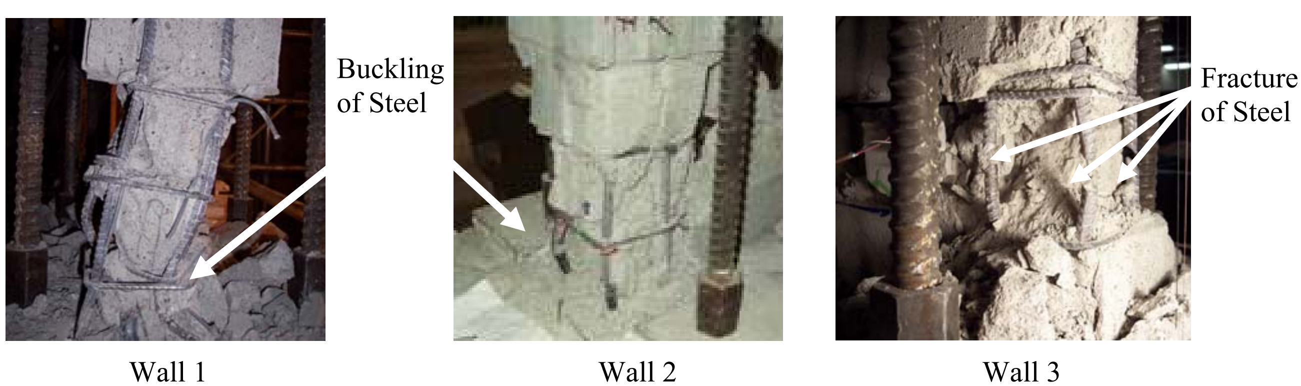

Figure 7: Wall Toe Damage after Failure

Overall failure of each wall was governed by buckling of the reinforcing and compression crushing of the grouted core or the breaking of reinforcing bars under tension. Figure 7 shows the heavily damaged boundary elements of each wall after failure has occurred. Based on the test results, masonry has shown to be capable of sustaining large drifts with minor strength degradation and high energy dissipation..

CONCLUSION

The presence of confinement at the ends of the walls tested demonstrated improved ductility and resilience to cyclic loading that is not seen in typical linear walls without confinement. The three walls tested had applied axial loads representative of a typical, lower bound and higher bound of realistic gravity loading cases experienced in real design. Each wall demonstrated high amounts of energy dissipation as well as ductile behaviour. Drift levels in excess of 2%, as well as a displacement ductility level (μ_Δ) of 10 was reached with no strength degradation in the walls with low to moderate axial stress. The wall with the high level of axial stress was able to sustain a drift of 1.4% and μ_Δ = 4 with no loss of capacity despite heavy damage encountered due to testing problems. There is an urgent need for similar tests in order to facilitate adoption of walls with boundary elements as a new masonry construction technique in building codes and by designers for seismic areas.

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