BEHAVIOUR OF PARTIALLY GROUTED REINFORCED MASONRY SHEAR WALLS-EXPERIMENTAL STUDY

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ABSTRACT

It is suggested that the conservative provisions of the Canadian masonry design standard for reinforcement details of masonry shear walls, restrict the use of reinforced masonry construction as a cost-efficient building system in regions with moderate seismic risks such as found in most parts of Canada. The experimental study presented in this paper was designed to provide information to evaluate the performance of partially grouted reinforced concrete block shear walls under in-plane cyclic loading. Five masonry shear walls having less than the commonly used minimum amount of reinforcing steel and larger bar spacing than specified in the seismic requirements of the current Canadian masonry design standard [1] were tested. The effect of aspect ratio (3 levels) and reinforcement spacing (3 levels) was considered within the test matrix. Wall specimens were constructed using half-scale model units of a 20-cm hollow concrete block. The ultimate load carrying capacities of the test walls indicated close agreement with the strengths predicted using the Canadian masonry standard. According to the proposed pseudo displacement ductility calculations, it was also found that the seismic load reduction factors suggested in the Canadian masonry design standard [1] underestimates the energy dissipation ability of partially grouted reinforced masonry shear walls despite large reinforcement spacing.

KEYWORDS: Shear Wall, Partially Grouted, Ductility, Experimental, Masonry, Reinforced, Concrete Block

INTRODUCTION

To ensure satisfactory response under significant earthquake excitation, masonry codes require designers to use reinforced masonry and comply with a set of minimum requirements for the amount and distribution of reinforcement within the concrete masonry units. CSA S304.1 [1] specifies that at sites where the seismic hazard index is equal to or greater than 0.35 (areas with seismic activity ranging from medium to high), loadbearing walls, including shear walls, must be reinforced horizontally and vertically with steel having a minimum total area of $0.002A_g$ (A_g corresponds to the gross area of the wall). The spacing between horizontal and vertical seismic reinforcement are also required not to exceed 1200 mm. These provisions of CSA S304.1 [1] are considered to be overly conservative compared to the similar provision specified in the MSJC [2] which allows engineers to use up to 3000 mm bar spacing for regions with low earthquake excitations. Although larger spacing (up to maximum 2400 mm) is permitted in Canada when the seismic hazard index is less than 0.35, there is very little data available to confirm satisfactory shear wall behaviour at spacing up to the permitted 2400 mm. Thus, designers have tended not to utilize spacing of more than 1200 mm in any seismic design of reinforced masonry.

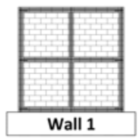
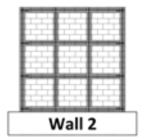
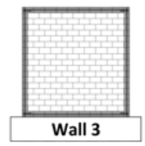
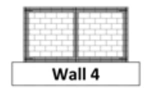
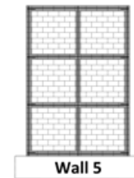
In this paper the performance of partially grouted reinforced masonry (PG-RM) shear walls, having less steel or larger spacing than specified as minimum seismic requirements is addressed. The primary objective of this study is to document the effects of reinforcement spacing and aspect ratio on the behaviour of PG-RM shear wall under reversed cyclic loading. The performance will be judged in terms of in-plane lateral load carrying capacity and post peak load characteristics such as energy dissipation and displacement ductility. The outcomes of the study are intended to provide a step forward towards modifying current prescriptive provision concerning minimum reinforcement requirements and load reduction factors for masonry structures in low or moderate seismic regions.

EXPERIMENTAL PROGRAM

The experimental program was designed to investigate the effects of wall aspect ratio (at three levels) and reinforcement spacing (at three levels) within five shear wall specimens. Half-scale model units of hollow 20-cm concrete blocks (actually 47 percent scale) were used in construction of the test specimens [7]. The matrix of the test program is presented in Table 1. As can be seen, all wall specimens were 1800 mm long (3600 mm long at full scale) with approximately the same steel ratio in the vertical and horizontal directions. Walls 1, 2 and 3 were intended to study the effect of reinforcement pattern. Wall 4 and Wall 5 along with Wall 1 were designed to investigate the effect of aspect ratio (height to length) at 0.5, 1.0 and 1.5. A constant superimposed axial load of 120 kN corresponding to a compressive stress of 0.75 MPa (based on gross area) was applied on each wall. The applied axial stress represents an axial loading likely to be resisted by a shear wall in a typical five-storey masonry building.

To simulate the effect of a relatively rigid foundation, each wall was constructed on a 500 mm wide by 400 mm high reinforced concrete base. Using half-scale hollow concrete blocks required the qualified mason to reduce the thickness of mortar joints to 5 mm. Standard type S mortar was used for wall construction with an average compressive strength of 21.4 MPa (C.O.V. 15.7%) for 50-mm mortar cube samples cast from each batch of mortar before using it for construction. Test walls were filled with fine grout (267 mm slump) at the locations of horizontal and vertical reinforcement. Since the walls were partially grouted, each wall was grouted in multiple lifts during construction. Three 200-mm-high by 102-mm-diameter cylinders and three block-moulded prisms using half-scale blocks with the approximate prism dimension of 90 mm × 90 mm × 185 mm were cast as control specimens during each lift of grouting. An average uniaxial compressive strength of 36.0 MPa (C.O.V. 10.9%) and 37.6 MPa (C.O.V. 14.2%) was obtained for the cylinders and prism grout samples, respectively.

Table 1: Test matrix for shear wall specimens

Specimen Label	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5
Reinforcing Pattern					
Dimensions	L = 1800 mm H = 1800 mm T = 90 mm	L = 1800 mm H = 1800 mm T = 90 mm	L = 1800 mm H = 1800 mm T = 90 mm	L = 1800 mm H = 900 mm T = 90 mm	L = 1800 mm H = 2700 mm T = 90 mm
Number and Size* of Reinf.	3 × #10 (V) 3 × D4 (H)	4 × #3 (V) 4 × D3 (H)	2 × #4 (V) 2 × 2D3 (H)	3 × #10 (V) 2 × D3 (H)	3 × #10 (V) 4 × D4 (H)
Reinforcement Ratio**	ρ_v = 0.19% ρ_h = 0.05%	ρ_v = 0.18% ρ_h = 0.05%	ρ_v = 0.16% ρ_h = 0.05%	ρ_v = 0.19% ρ_h = 0.05%	ρ_v = 0.18% ρ_h = 0.04%
Bar Spacing	855 mm	570 mm	1710 mm	855 mm	855 mm
Aspect Ratio	1.0	1.0	1.0	0.5	1.5
Axial Stress**	0.75 MPa	0.75 MPa	0.75 MPa	0.75 MPa	0.75 MPa

* *See Table 2*

** *Based on gross area*

The vertical reinforcement used in this experimental program include Canadian No. 10, USA No. 3 and No. 4 bars. For horizontal reinforcement, D3 and D4 deformed wires were used. Three randomly chosen 600-mm-long samples were tested under uniaxial tension for each bar size. The geometric and mechanical properties of the bars used in the construction of the shear wall specimens are listed in Table 2.

Six hollow masonry prisms and six grouted masonry prism were fabricated during construction of each wall specimen in order to identify the uniaxial compressive strength and corresponding strain. The average uniaxial compressive strength of grouted prisms was 12.4 MPa (C.O.V. = 6.8%) corresponding to an average strain of 0.0013 (C.O.V. = 15.1%). An average uniaxial compressive strength of 21.6 MPa (C.O.V. = 4.7%) was also obtained for hollow prisms with an average strain of 0.0013 (C.O.V. = 18.6%) at maximum stress.

TEST SETUP

A sketch of the setup used for the shear wall tests is illustrated in Figure 1. The setup was intended to provide lateral and axial in-plane loading with the optimal possible accuracy.

Table 2: Mechanical properties for reinforcing steel bars used in shear walls

Type	A_s (mm^2)	E_s (GPa)	f_y (MPa)	ε_y (mm/mm)
CDN No. 10	100	201.6	491.7	0.0024
USA No. 3	71	201.0	502.9	0.0025
USA No. 4	126	201.8	564.7	0.0028
D3	19.4	183.6	743.7	0.0041
D4	25.8	198.2	690.7	0.0035

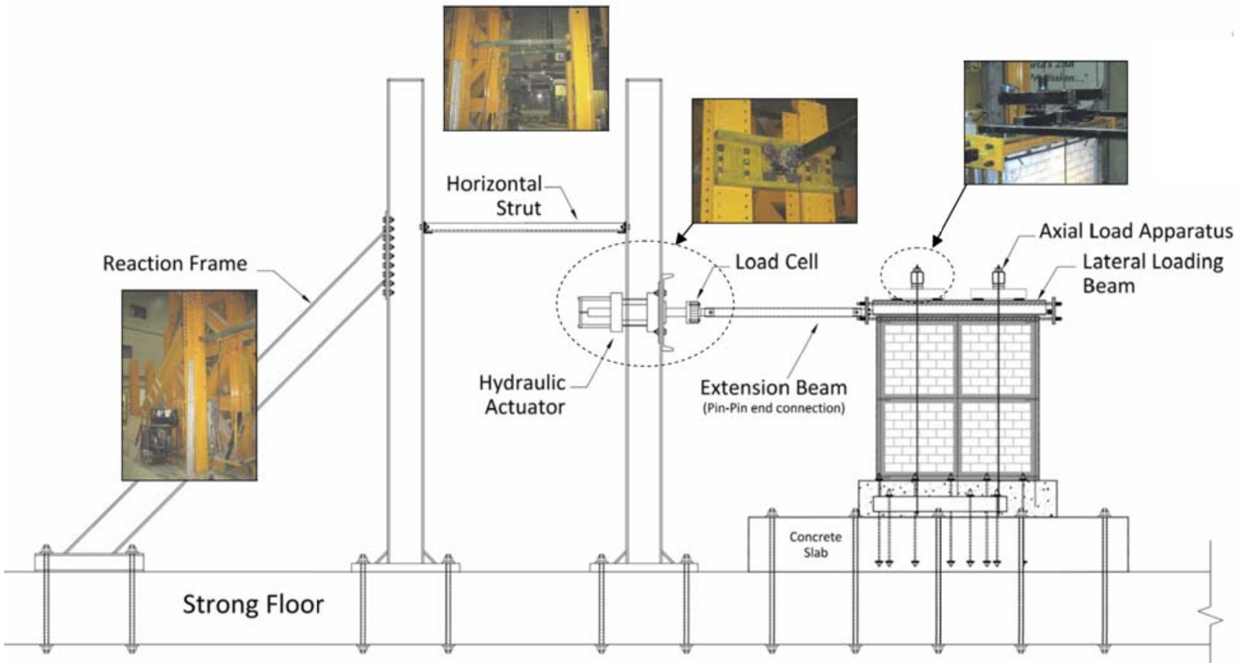


Figure 1: Front view of the test setup

A 600- mm-deep reusable concrete slab**,** which had been prestressed down to the 600-mm-thick strong floor of the laboratory**,** was used as a rigid platform for testing. The base of the wall specimen was prestressed down onto this testing platform. A hydraulic actuator was used to apply the lateral load on the wall specimens. A steel beam, as shown in Figure 2, was attached to the top of the wall and used to transfer the applied lateral load uniformly along the top of the wall. The vertical bars were extended above the top of the wall and were welded to the loading beam. To help ensure uniform shear transfer from the loading beam to the wall, the beam was also anchored to the wall with four pairs of 75×100×12 mm steel angles and extra 150 mm lengths of No. 10 bar anchored in the grouted top course (see Figure 2). The contact surfaces between the steel angles and the test wall as well as the holes for No. 10 bars were coated with a layer of a commercial high strength structural adhesive before installation. When the adhesive hardened**,** the angles were welded to the loading beam and the anchoring bars.

The axial load was applied to the top of the wall with two hollow cylindrical hydraulic jacks operated by manual pumps. To ensure uniform distribution of axial load, the axial load was transferred to the top loading beam at the one-fifth points along the length of the wall.

The loading beam (and the wall) was braced against potential out-of-plane deflection using two transverse steel box sections attached to steel columns and hinged to the top loading beam by a pair of rollers.

INSTRUMENTATION

Lateral displacements of the walls at the upper left and upper right corner of the wall were monitored during the tests using dial gauges mounted at designated heights. The dial gauges at each end were attached to a stiff reference frame that was laterally braced to prevent movement.

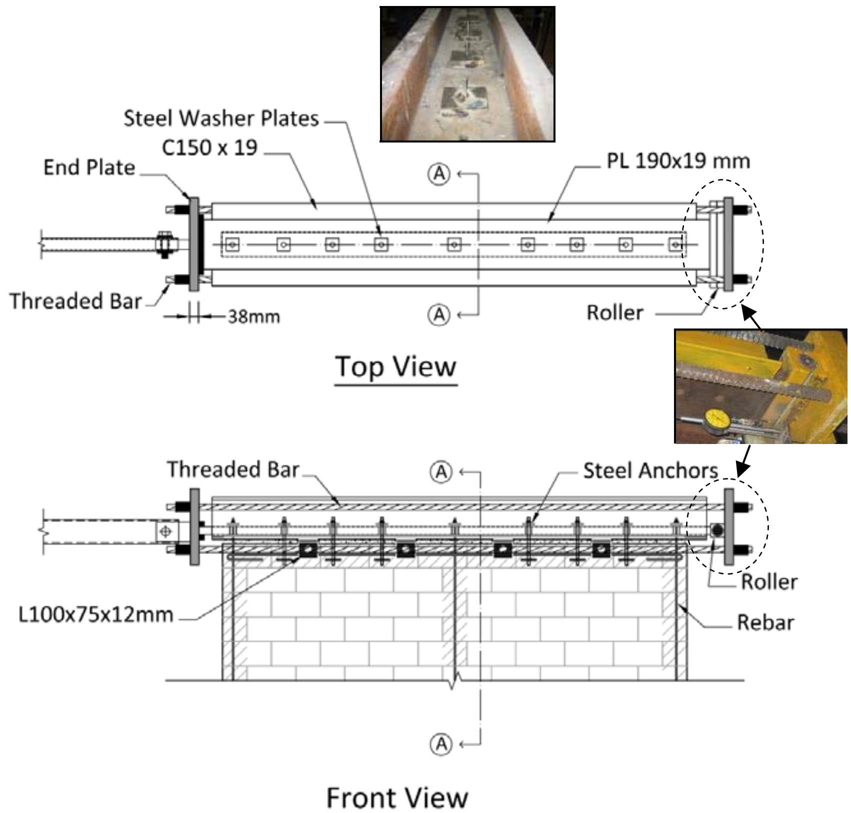


Figure 2: Details of loading beam for shear wall tests

(It should be noted that electronic displacement measurements were not found to be sufficiently accurate for the very small displacements occurring in this test.) Six electrical strain gauges were also attached to vertical reinforcing bars at the ends of the walls to monitor the strain history of these outermost steel bars during the test.

LOADING PROCEDURE

All test walls were subjected to fully reversed cyclic loading controlled by displacements measured in the push and pull directions. Loading started with a displacement amplitude corresponding to approximately 0.01% storey drift and then was increased progressively by approximately 150% of the previous cycle depending on the damage progression observed during each cycle. Each loading cycle was repeated twice and testing continued until the remaining resistance had degraded by more than 50% of the ultimate strength of the test specimen.

GENERAL EXPERIMENTAL OBSERVATIONS

The load-displacement hysteresis loops of the test walls are presented in Figure 3. Figure 4 shows the crack pattern for Wall 1 as a typical partially grouted reinforced masonry shear wall. In general, cracking was visible at about 50% of the peak load in the form of horizontal bed joint cracks taking place within the lower one-third of all five shear wall specimens. The horizontal cracks extended towards the centre of the test wall up to approximately 85% of the peak load. Diagonal stepped pattern cracks appeared at about 70% of the maximum load carrying capacity

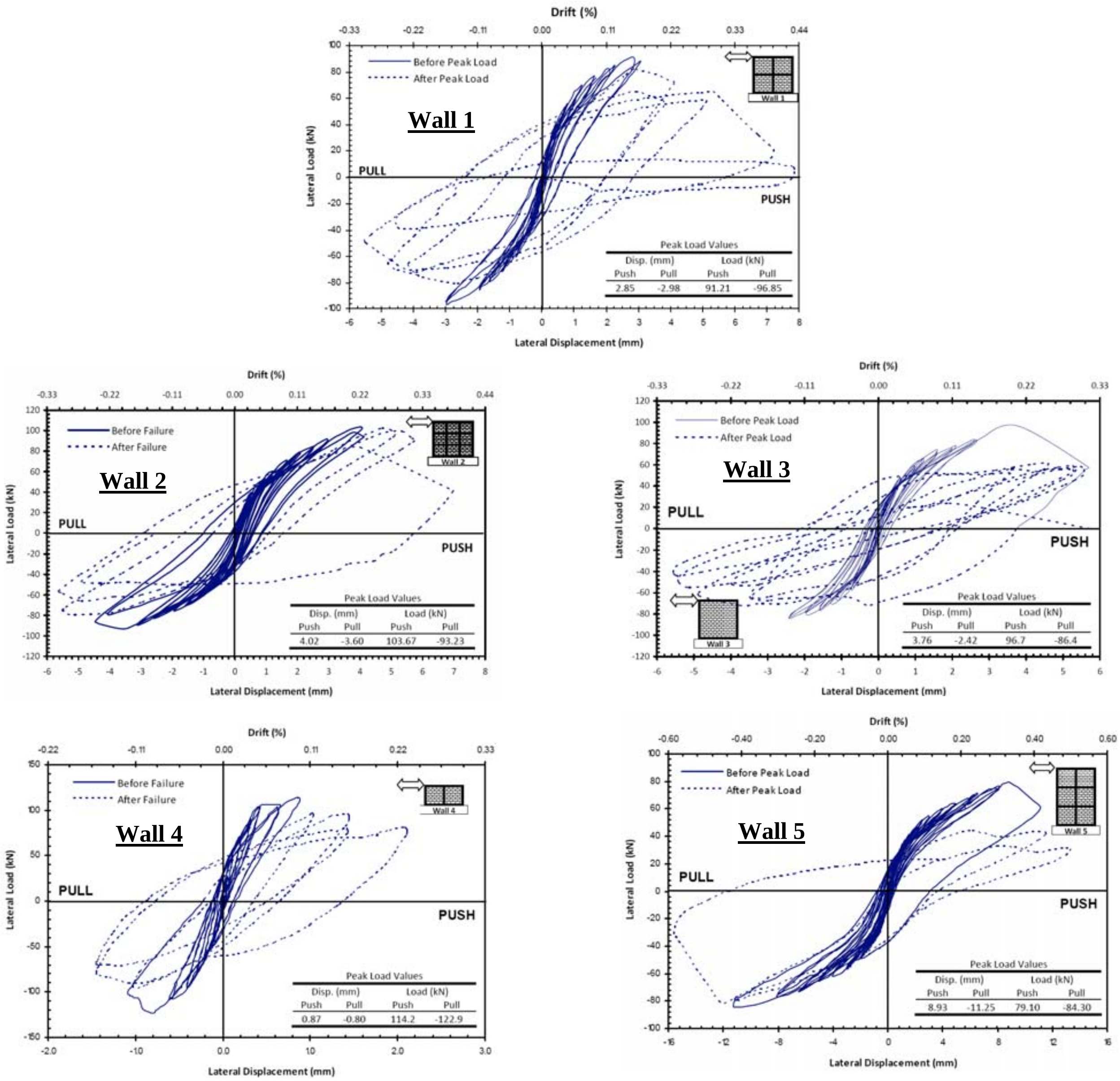


Figure 3: Load-displacement hysteresis loops

of the walls. Upon reaching the peak load, diagonal cracks penetrated into the concrete blocks in the high compression zones at the toe of the wall followed by toe crushing. The strength and lateral stiffness of the wall gradually decreased with increased displacement amplitude of the loading cycles. The vertical reinforcing bars embedded in the outermost grouted cells were found to be permanently deformed in all test walls due to inelastic buckling. Testing continued until the remaining resistance of the wall degraded by at least 50% of the peak load. Based on strains recorded by three strain gauges attached to vertical reinforcing steel bars**,** tensile yielding was not observed for either end bar of the test walls except for Wall 5 (with an aspect ratio of 1.5) where the vertical tensile bar yielded at about 0.24% storey drift in the push and pull directions. This indicates a shear dominated type of failure for the first 4 walls having an aspect ratio equal to or less than 1.0 and a mixed shear-flexure failure [mode] for Wall 5 with an aspect ratio of 1.5.

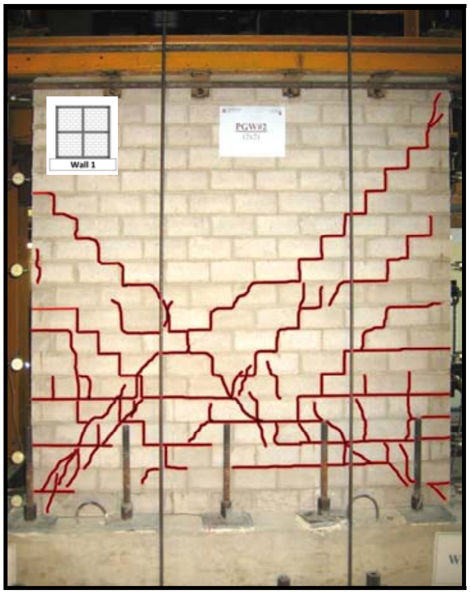
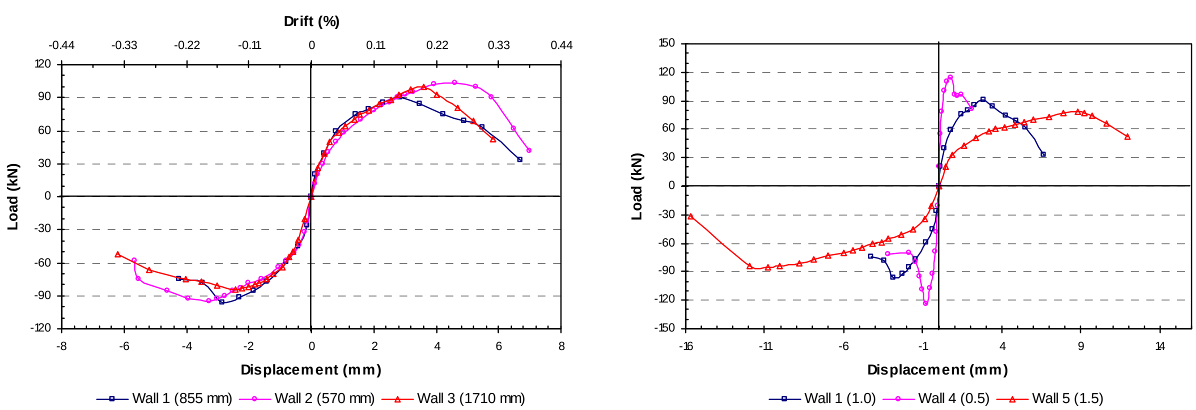


Figure 4: Crack pattern for Wall 1 at 60% of the ultimate strength after the peak load (Photos enhanced for crack pattern.)

GENERAL LOAD DISPLACEMENT RESPONSE OF THE TEST WALLS

The overall responses of shear walls are compared in Figure 5 using the envelopes of the load-displacement hysteresis loops recorded during the tests. As can be seen in Figure 5-a, Walls 1, 2 and 3, having the same aspect ratio and amount of vertical and horizontal steel**,** showed fairly similar behaviour under the imposed lateral loading cycles. This indicates that the global shear dominated responses of the test walls were not very sensitive to the reinforcement pattern. In contrast, as shown in Figure 5-b, Walls 4, 1 and 5 with aspect ratios of 0.5, 1.0 and 1.5, respectively, experienced significantly reduction in maximum load carrying capacity (shear capacity) and stiffness with increased height. However, the maximum base moments reached at the peak load increased significantly but not proportionally with the increase in aspect ratio. This indicates shear rather than flexure dominated capacity since flexural capacity should be constant for the same cross section.



(a) Walls with an aspect ratio of one and different bar spacing

(b) Walls with different aspect ratios and similar bar spacing

Figure 5: Load-displacement envelope of the test walls

CAPACITY

A summary of the predicted and measured capacities (maximum lateral load resistance) for the test walls is presented in Table 3. The flexural and shear capacities of the walls were calculated based on the actual material properties for masonry and steel measured via auxiliary tests. According to the code requirements (CSA S304.1), the maximum usable compressive strain, ε_{mu}, at the extreme masonry fibre was assumed to be 0.003. As can be seen in Table 3, two sets of calculations were conducted for shear capacity of the walls; one assuming 100% contribution of horizontal shear reinforcement and the other one assuming only 60% as per CSA S304.1 [1].

Shear strength was predicted to be lower than the flexural strength for Walls 1, 2, 3 and 4 which indicates domination of shear failure in these tests. This is consistent with the test observations where, except for Wall 5, none of the test walls experienced vertical bar yielding before reaching ultimate (peak) load. Wall 5 could be characterized as having a mixed shear-flexure failure mode because both tensile yielding of vertical reinforcement, and diagonal cracking and eventual failure through the compression zone were clearly observed during the test. Comparisons between calculated shear strengths and measured ultimate loads indicated very close agreement between predicted and measured strength values. Wall 5, with yielding of flexural steel, also shows very close agreement with the strength prediction using CSA [1] provisions where the predicted lateral loads for flexural and shear capacities were relatively close. Calculated first yield capacities for the other four walls were higher than their calculated shear capacities which were consistent with the test results.

Table 3: Predicted and measured values for the ultimate load

Specimen No. Bar Spacing (Aspect Ratio)	Predicted (kN)		Measured (kN)		Predicted / Measured*
	Flexural Strength	Shear Strength 60% Hor. Steel (100% Hor. Steel)	Push (Drift)	Pull (Drift)	
Wall 1 855 mm (1.0)	121.1	88.3 (102.9)	91.2 (0.16%)	96.9 (0.17%)	0.94 – SH 60%** 1.09 – SH 100%
Wall 2 570 mm (1.0)	115.9	97.6 (115.3)	103.7 (0.22%)	93.2 (0.20%)	0.99 – SH 60% 1.17 – SH 100%
Wall 3 1710 mm (1.0)	122.2	79.4 (91.2)	96.7 (0.21%)	84.4 (0.13%)	0.88 – SH 60% 1.01 – SH 100%
Wall 4 855 mm (0.5)	230.6	103.8 (115.7)	114.2 (0.10%)	122.9 (0.09%)	0.88 – SH 60% 0.98 – SH 100%
Wall 5 855 mm (1.5)	80.7	88.3 (102.9)	79.1 (0.33%)	84.3 (0.42%)	0.99 – F

* *Average strength of the wall in the push and pull directions was considered.*

** *SH 60/100%: Shear dominated failure mode assuming 60/100% effectiveness for horizontal steel, F: Flexural dominated failure mode*

DUCTILITY RELATED FORCE MODIFICATION FACTORS

Even though shear failures dominated, the data shows a significant ductility or pseudo ductility. In order to evaluate the inelastic performance of the test walls, the hysteretic behaviours of the walls were analyzed using idealised bilinear resistance envelopes based on an equal energy approach suggested by Tomazevic [4]. This approach and associated parameters are defined in Figure 6. The ultimate resistance, H_u, was assumed to be 90% of the maximum lateral load carrying capacity of the wall [4]. The ultimate idealised displacement, d_u, was defined where

20% strength degradation occurred in the experimental load-displacement curve [4]. The force modification factor, calculated based on the equal energy concept [5,6], is defined by:

$$R_\Delta = \sqrt{2\mu_\Delta - 1} \tag{2}$$

where μ_Δ is displacement ductility of the structure defined by $\mu_\Delta = d_u / d_e$. Parameter d_e is the displacement at the idealised elastic limit. The force modification factors, calculated based on the described method, are presented in Table 4. It can be seen from Table 4 that all force modification factors are greater than 2.0 which is the maximum R_Δ value currently specified in CSA S304.1 [1] for buildings with moderately ductile masonry shear walls. This indicates that the seismic force modification factor, R_Δ, suggested by the CSA S304.1 [1], underestimates the energy dissipation ability of partially grouted reinforced masonry shear walls despite large reinforcement spacing and even no yielding of the tension reinforcement

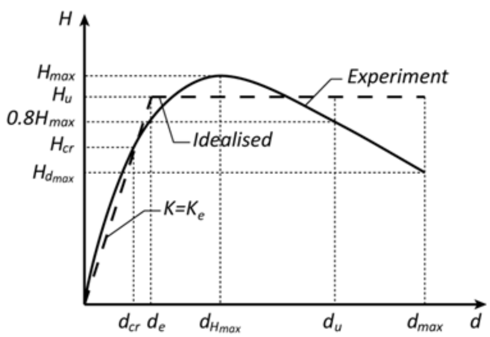


Figure 6: Idealisation of experimental hysteresis envelope with bilinear relationship [4]

Table 4:Force modification factors for test walls

Specimen No. Bar Spacing (Aspect Ratio)	Calculated Force Modification Factor* (R)	
	Push	Pull
Wall 1 855 mm (1.0)	2.8	2.4
Wall 2 570 mm (1.0)	2.6	3.0
Wall 3 1710 mm (1.0)	2.4	3.2
Wall 4 855 mm (0.5)	3.4	2.5
Wall 5 855 mm (1.5)	2.6	2.5

* *Based on equal energy approach.*

CONCLUSIONS

The cyclic responses of the partially grouted reinforced masonry shear walls under in-plane axial and lateral loading were addressed with the main focus being on the effect of reinforcement spacing and aspect ratio of the walls. It was found that the overall response of the test walls is not sensitive to the reinforcement pattern whereas it is sensitive to the aspect ratio of the wall. Close agreement was observed between the shear strengths calculated using the Canadian masonry design standard [1] and the experimental results. The extension of these findings to cover very different combinations of reinforcing and axial load may require additional evaluations. However, based on the results described in this paper, a value greater than or equal to 2.4 was obtained for the seismic force modification factor, R_Δ, or all test walls. This indicates that for similarly partially grouted walls with widely spaced reinforcement, an R_Δ value of at least 1.5 or more would be appropriate. From these results and other tests at McMaster University [8] on fully grouted walls with more closely spaced reinforcing, it also seems clear that the R_Δ values of 1.5 and 2.0, suggested by the Canadian masonry design standard [1] for walls identified as having limited ductility or being moderately ductile, respectively, underestimate their energy dissipation ability. Even partially grouted reinforced masonry shear walls with the shear dominated behaviour observed for the test walls indicate greater ductility. Therefore**,** it is suggested that the current R_{Δ} values for reinforced masonry shear walls specified in the current code need to be re-examined with the view to using higher values and of including partially grouted bars with widely spaced reinforcing as another wall type for regions of low to moderate seismic activity.

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