

In-Plane Seismic Performance of Fully Grouted Reinforced Masonry Shear Walls

Hany M. Seif EIDin, M.ASCE¹; and Khaled Galal, M.ASCE²

Abstract: Reinforced masonry (RM) shear walls are key structural elements widely used to resist lateral loads in masonry buildings due to their capability to provide lateral strength, stiffness, and energy dissipation. This paper investigates the in-plane seismic performance of fully grouted RM shear walls dominated by shear failure. The experimental work involved assessing the response of five single-story RM shear walls when subjected to in-plane axial compressive stress, cyclic lateral excitations, and top moment. The studied parameters were the horizontal reinforcement ratio, level of axial compressive stress, and shear span to depth ratio. Most of the current provisions in masonry design codes for nominal in-plane shear strength, V_n , for RM shear walls consider a percentage of the yield capacity for the shear resistance provided by the horizontal reinforcement, V_s . Moreover, they limit the effect of the shear span to depth ratio, M/Vd_v , to an upper value of 1.0. The test results concluded that the horizontal reinforcement could contribute 100% of its yield capacity toward V_n . In addition, increasing M/Vd_v from 1.25 to 1.875 resulted in a significant reduction in the shear strength by 25%, which means that limiting the effect of M/Vd_v to an upper value of 1.0 is overestimating V_n of RM shear walls at high values of M/Vd_v . However, this reduction is accompanied by higher displacement ductility. On the other hand, increasing the axial compressive stress resulted in a higher V_n with more brittle failure. DOI: 10.1061/(ASCE)ST.1943-541X.0001758. © 2017 American Society of Civil Engineers.

Author keywords: Reinforced masonry; Shear walls; Cyclic loading; Horizontal reinforcement; Shear-span to depth ratio; Concrete and masonry structures.

Introduction

The poor performance of unreinforced masonry buildings subjected to earthquake ground motions has led to the development of reinforced masonry (RM) systems. Similar to reinforced concrete (RC) buildings, RM shear walls are key structural elements used to resist lateral loads in regions of high seismic or wind loads. Reinforced masonry shear walls must be capable of providing the required lateral strength, stiffness, and energy dissipation in these regions. The flexural behavior of RM shear walls is well-defined and follows the simple flexural theory of RC structures based on plane-section assumption. On the other hand, the shear behavior of RM shear walls in the plastic hinge region is more complex due to the lack of comprehensive models that quantify the interaction of various mechanisms that contribute to shear capacity. Many experimental studies have been conducted to investigate the in-plane shear performance of full-scale RM shear walls under cyclic lateral loading (e.g., Sveinsson et al. 1985; Kaminosono et al. 1988; Matsumura 1986; Shing et al. 1990; Voon and Ingham 2006). Additional studies have been carried out using scaled blocks such as those by Tomazevic et al. (1996), Haach et al. (2010), and El-Dakhkhni et al. (2013). Using the best-fit analysis for experimental test results, in-plane shear-strength equations have been developed by different researchers and standards. Seif EIDin and Galal (2015)

conducted a survey of the existing design equations for the nominal in-plane shear strength, V_n , for RM shear walls. Their study concluded that the main parameters that control V_n could be summarized as wall cross section dimensions, compressive strength of masonry, f'_m , shear span to depth ratio, M/Vd_v , axial compressive stress, σ_n , vertical and horizontal reinforcement ratios and yield strength, and the achieved displacement ductility, μ_Δ . Anderson and Priestley (1992) proposed the first effective equation that considered V_n as the sum of three independent terms provided by masonry, V_m , axial compression load, V_p , and horizontal reinforcement, V_s , as follows:

$$V_n = V_m + V_p + V_s \quad (1)$$

Most previous researchers did not experimentally measure the resistance provided by each component, V_m , V_p , and V_s , separately, but rather quantified them based on the difference between the achieved maximum lateral loads for tested RM shear walls with varied studied parameters. This quantification approach overlooks the effect of the redistribution in the shear resistance shares between them, especially at high levels of displacement ductility. As a result of this previous evaluation approach, most of the existing design equations for V_n for RM shear walls including the current provisions of the Canadian Standards CSA S304 (CSA 2014b), the U.S. Masonry Standards Joint Committee (MSJC 2013), and the New Zealand Standards NZS 4230 (NZS 2004) for masonry structures consider a percentage of the yield capacity for the in-plane shear resistance provided by the horizontal reinforcement, V_s . Moreover, they limit the effect of the shear span to depth ratio, M/Vd_v , to an upper value of 1.0. Therefore, this paper aims to investigate the effect of the horizontal reinforcement ratio, ρ_h , level of axial compressive stress, σ_n , and shear span to depth ratio, M/Vd_v , on the in-plane shear behavior of RM shear walls by measuring experimentally the shear resistance provided by the horizontal reinforcement, V_s , consequently, V_{m+p} .

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Table 1. Test Matrix of Five RM Shear Walls

Wall identifier	Reinforcement		Studied parameter			Theoretical capacities according to CSA S304-14		
	Vertical	Horizontal	ρ_h (%)	σ_n (MPa)	M/Vd_v	F_{Vn}^a (kN)	F_{Mn}^b (kN)	F_{Sn}^c (kN)
W-Ref	20M@200	10M@400	0.13	1.0	1.2	269	541	1469
W- ρ_h 0	20M@200	—	0.0	1.0	1.2	177	541	1469
W- σ_n 0	20M@200	10M@400	0.13	0.0	1.2	212	455	1161
W- σ_n 1.5	20M@200	10M@400	0.13	1.5	1.2	298	576	1623
W-M/Vd _v 1.8	20M@200	10M@400	0.13	1.0	1.8	269	361	1469

^aTheoretical in-plane shear capacity assuming moderately ductile shear walls ($R_d = 2.0$).
^bTheoretical lateral force corresponding to the flexural capacity assuming plane sections assumption.
^cTheoretical lateral force corresponding to sliding capacity.

Experimental Work

The experimental work assessed the response of five full-scale fully grouted rectangular RM shear walls when subjected to in-plane axial compressive stress, cyclic lateral excitations, and top moment. The effect of the horizontal reinforcement ratio, ρ_h , the level of axial compressive stress, σ_n , and the shear span to depth ratio, M/Vd_v , on their in-plane shear behavior was considered in the test matrix as presented in Table 1. Moreover, the table includes the theoretical lateral forces corresponding to the in-plane shear, flexural, and sliding capacities according to the Canadian Standards CSA S304 (CSA 2014b). All the tested walls had the same dimensions of $1.8 \times 1.6 \times 0.19$ m and were vertically reinforced with 20M bar in each cell with a flexural reinforcement ratio, ρ_v , of 0.79%, as shown in Fig. 1. The first wall, W-Ref, was designed to be a reference wall for the remaining tested walls. Wall W-Ref was constructed with uniformly distributed horizontal reinforcing bars, 10M at 400 mm, which were hooked using the standard 180° bend around the outermost wall flexural reinforcing bars. A constant axial compressive stress of 1.0 MPa was applied to the studied wall, W-Ref, before subjecting it to in-plane cyclic lateral displacements and it remained constant during the whole test.

Wall W- ρ_h 0 was built without horizontal reinforcement. Thus, comparing the behavior of Walls W-Ref and W- ρ_h 0 enabled quantifying the horizontal reinforcement contribution, V_s , towards the nominal shear strength, V_n , and its effect on the displacement ductility of tested walls. In addition, the impact of the horizontal reinforcement on the redistribution of the shear resistances provided by the masonry and axial compressive stress, V_{m+p} ,

was investigated. In order to evaluate the effect of the axial compressive stress, σ_n , on the in-plane shear behavior of RM shear walls, Walls W- σ_n 0 and W- σ_n 1.5 were duplicates of Wall W-Ref, but with external σ_n of 0.0 and 1.5 MPa, respectively, instead of 1.0 MPa for Wall W-Ref. One of the important parameters that affects the inelastic behavior and ductility of RM shear walls is the shear span to depth ratio, M/Vd_v . Wall W-M/Vd_v1.8 was subjected to a top moment of 0.9V kN · m, where V is the lateral load from the horizontal actuator, which resulted in an overall M/Vd_v equal to 1.875 as compared to a value of 1.25 for Wall W-Ref.

Properties of Materials

To provide a fixed end condition at the base of tested walls, each wall was constructed on a strong RC foundation. The vertical reinforcement of each wall was bent and anchored in the RC foundation at the base and was threaded for a length of 100 mm from the top to connect tested walls to the top steel loading beam. All walls and the required auxiliary specimens were constructed using light-weight knockout concrete masonry units (CMUs) with nominal dimensions of $390 \times 190 \times 190$ mm. These knockout units are masonry units that have knockout webs, which can be removed to accommodate the horizontal reinforcement. In addition, these types of units were selected to provide grouting continuity in the vertical and horizontal directions, consequently, preventing any weakness planes between the concrete masonry units. The blocks were joined together with 10-mm Type-S mortar joints and laid in a running bond pattern. All the walls were fully grouted using coarse grout with a maximum aggregate size of 6.35 mm (1/4-in.) that was mixed in the laboratory in accordance with CSA A179 (CSA 2004). Table 2 summarizes the properties of the construction materials.

Test Setup

Three MTS hydraulic actuators were used to apply the in-plane loads as shown Fig. 2. Two actuators were installed vertically to apply the axial compression force and the top moment (when applicable) that act on the top of the wall. Moreover, the horizontal actuator was used to introduce cyclic horizontal displacements to simulate the seismic loads. The RC base foundation of the wall specimens was connected to a strong floor using a reusable RC footing with a depth of 600 mm to provide a fixed boundary condition at the base of the tested walls. The applied loads were transferred to the tested walls through a stiff built-up steel-loading beam. The three actuators were synchronized to apply the shear force, axial load, and top moment that represent the acting loads on the plastic hinge panel of the whole RM shear wall. Two

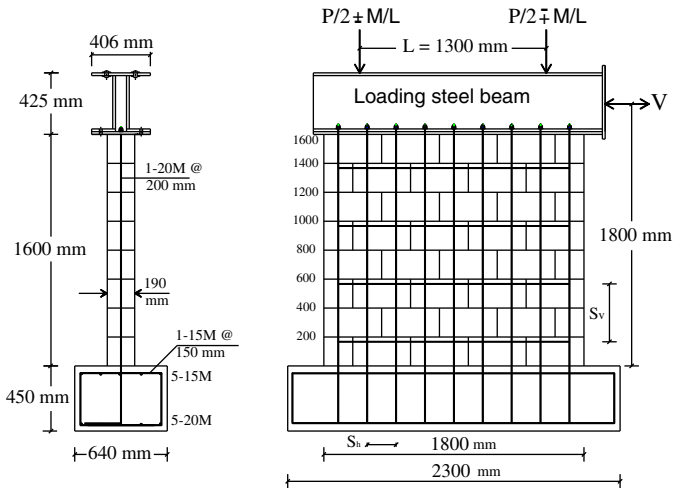
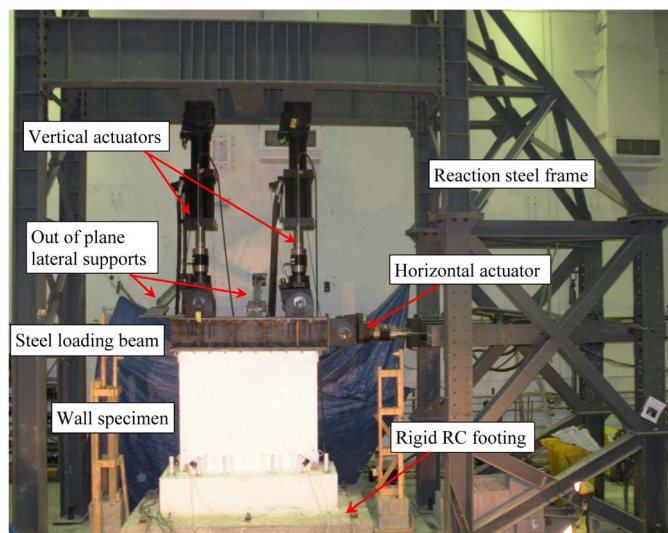


Fig. 1. Dimensions of a typical tested RM shear wall

Table 2. Properties of Construction Materials

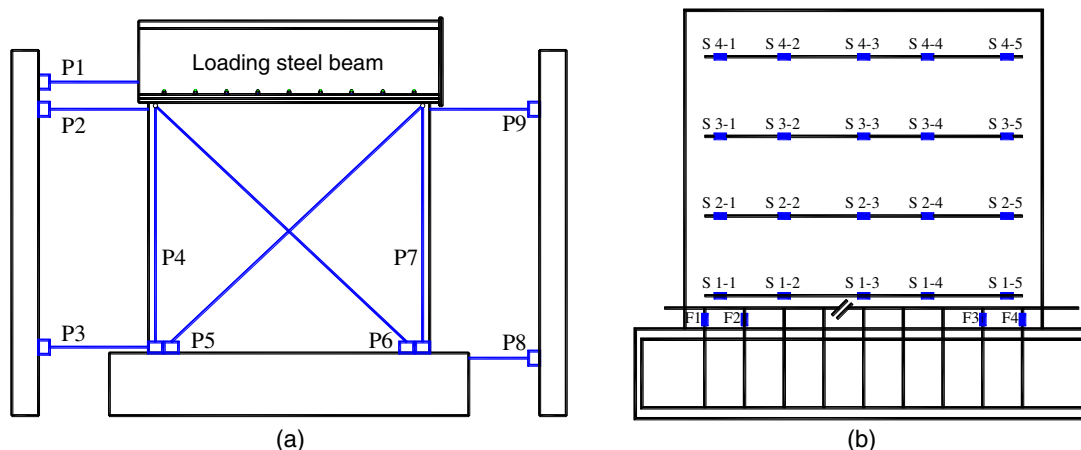
Material	Characteristic	Strength (coefficient of variation)	Reference
Type S mortar	Cube compressive strength, f'_{mo}	7 d: 8.5 MPa (8.8%) 28 d: 13.7 MPa (7.8%)	CSA A179-04 (CSA 2004)
Knockout concrete block	Flow table test	112.16% (3.8%)	ASTM C1437-13 (ASTM 2013b)
	Weight	12.9 kg (3.1%)	—
	Block compressive strength, f'_b	16.7 MPa (4.8%)	CSA A165.1-14 (CSA 2014a)
Footing concrete	Cylinder compressive strength, f'_c	7 d: 30.5 MPa (5.1%) 28 d: 39.5 MPa (6.6%)	ASTM C39M-12 (ASTM 2012a)
	Splitting tensile strength, f_r	28 d: 4.2 MPa (4.4%)	ASTM C496M-11 (ASTM 2011)
Coarse grout	Cylinder compressive strength, f'_g	7 d: 21.6 MPa (8.4%) 28 d: 29.4 MPa (7.3%)	ASTM C476-10 (ASTM 2010a) CSA A179-04 (CSA 2004)
Steel reinforcement	Yield strength, f_y	430 MPa (3.2%)	ASTM A615M-13 (ASTM 2013a)
	Modulus of elasticity, E_s	196 GPa (1.85%)	A370-14 (ASTM 2014)
	Ultimate strength	536 MPa (2.7%)	
Four-course running bond prism	Prism compressive strength, f'_m	13.1 MPa (7.6%)	ASTM C1314-12 (ASTM 2012b)
	Axial strain at maximum strength, ϵ_o	0.0025 (11.4%)	CSA S304-14 (CSA 2014b)
	Modulus of elasticity, E_m	6.9 GPa (17.4%)	
Seven-course stack bond beam	Prism tensile strength, f_t	1.8 MPa (9.5%)	ASTM E518-10 (ASTM 2010b)
	Tensile strain at maximum strength, ϵ_t	0.00046 (13.3%)	CSA S304-14 (CSA 2014b)

**Fig. 2.** Test setup for RM shear wall

out-of-plane steel back-to-back angles connected the loading steel beam (through slotted holes) to a strong RC wall to prevent any out-of-plane displacement.

Instrumentation and Testing Protocol

Two different types of instruments were used to monitor the deformations of the tested walls during testing. Nine linear potentiometers were attached externally to each of the tested RM shear walls to measure the horizontal, vertical, and diagonal displacements as shown in Fig. 3(a). The target displacement was measured as the difference between the average reading of Potentiometers P2 and P9 and the movement in the base foundation, P8, if any. The shear sliding displacement was calculated as the difference in the readings between P3 and P8. Potentiometers P4 and P7 were attached to the tested walls to measure the flexural tensile and compressive deformations, while P5 and P6 were used to measure the overall diagonal shear deformations. P1 was attached to the top loading steel beam to monitor any sliding between the loading beam and the tested wall.

**Fig. 3.** Locations of (a) displacement potentiometers; (b) strain gauges

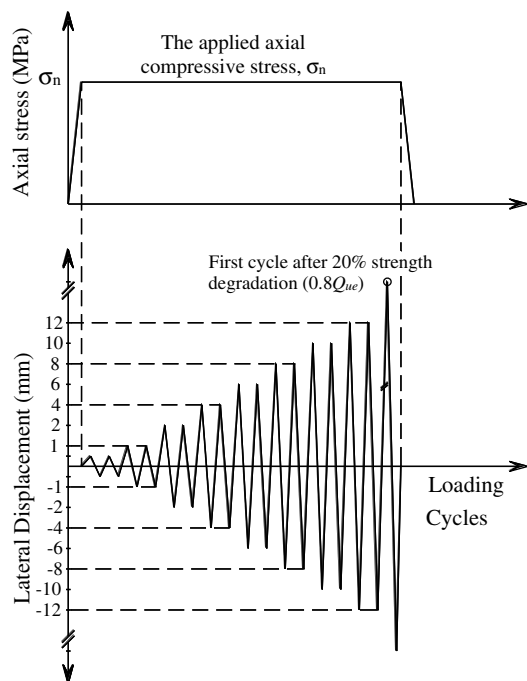


Fig. 4. Loading protocol

In addition to the potentiometers, 5-mm strain gauges were installed on the steel reinforcement bars to record the local strains during loading of the walls as shown in Fig. 3(b). Four strain gauges were installed at the wall–footing interface of the two

outermost vertical reinforcement bars on each side, to define the yield load. To measure the axial strain distribution in the horizontal reinforcement, five 5-mm strain gauges were installed and equally distributed along the total length of each bar. Using the experimentally measured stress-strain relationship and the cross-section area for the steel reinforcing bars, the average tension force in each bar was calculated and summed to measure the horizontal reinforcement contribution, V_s , to the in-plane shear strength, V_n .

The loads were applied in two phases. In the first phase, the total axial compression load was applied using a load-control protocol. Next, the test protocol was switched to displacement-control. In this second phase, in-plane lateral displacements were introduced at the midheight of the loading steel beam, according to the loading histories proposed by FEMA 461 (FEMA 2007) as shown in Fig. 4. In each stage of lateral loading, two displacement cycles were completed for each target displacement increment. The yield load was determined when the strain gauge located at the wall–footing interface reached the yield strain level. However, according to FEMA P-58 (FEMA 2009), the failure was defined as the point on the loading curve where the lateral resistance dropped to 80% of the maximum lateral load recorded, in whichever direction this occurred first.

Test Results

The test results indicated that all five walls exhibited shear-dominated response with diagonal cracks. Fig. 5 presents the hysteretic force-displacement response for each wall against the top drift, while Table 3 summarizes the results for tested walls in both push and pull directions including the crack, yield, and ultimate capacities in addition to the top drift of 1%. All the tested walls

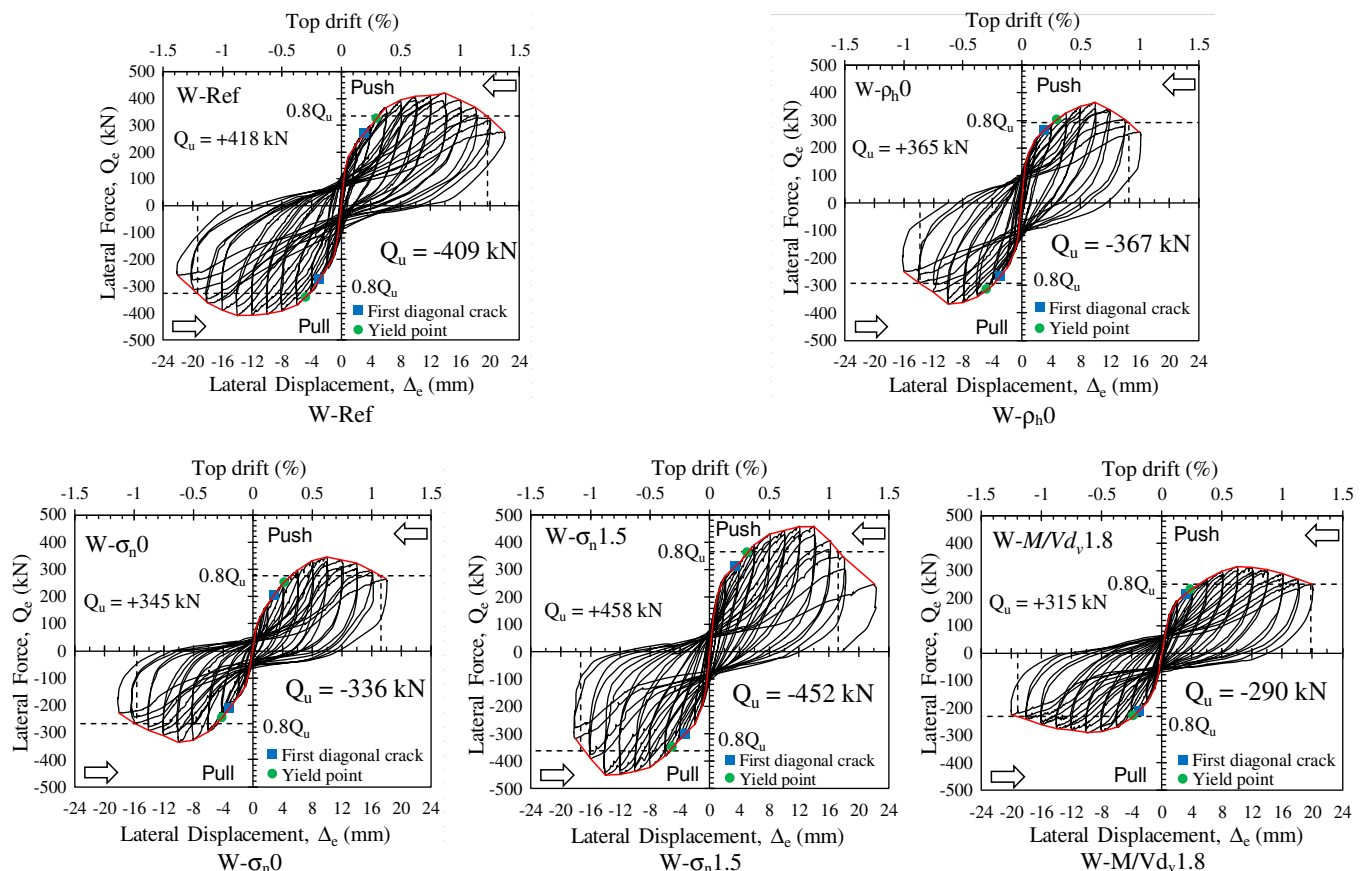


Fig. 5. Hysteretic force-displacement response of tested walls

Table 3. Summary of Experimental Force and Displacement Capacities of Tested Walls

Wall identifier	Test results									
	Yield		Peak		Failure	Displacement ductility			First crack ^a	
	Q_{ye} (kN)	Δ_{ye} (mm)	Q_{ue} (kN)	$\Delta_{Q_{ue}}$ (mm)	$\Delta_{0.8Q_{ue}}$ (mm)	$\mu_{\Delta_{Q_{ue}}}$	$\mu_{\Delta_{0.8Q_{ue}}}$	$\mu_{\Delta_{e1\%}}$	Δ_{FCe} (mm)	Q_{FCe} (kN)
W-Ref	328	4.7	418	14.0	19.7	3.0	4.2	3.4	3.2	272
	-339	-4.7	-409	-14.0	-19.4	3.0	4.1	3.4	-2.9	-274
W- ρ_h0	309	4.8	365	10.0	14.6	2.1	3.0	3.3	3.1	265
	-315	-4.8	-367	-10.0	-13.8	2.1	2.9	3.3	-3.0	-269
W- σ_n0	254	4.2	345	10.0	17.3	2.4	4.1	3.8	3.0	209
	-244	-4.2	-336	-10.0	-15.7	2.4	3.7	3.8	-3.1	-212
W- $\sigma_n1.5$	365	5.0	458	14.0	17.2	2.8	3.4	3.2	3.4	312
	-349	-5.0	-452	-14.0	-17.3	2.8	3.4	3.2	-3.3	-306
W-M/Vd $_v1.8$	230	3.8	315	10.0	19.8	2.6	5.2	4.2	3.2	216
	-225	-3.7	-290	-10.0	-19.2	2.7	5.3	4.4	-3.0	-210

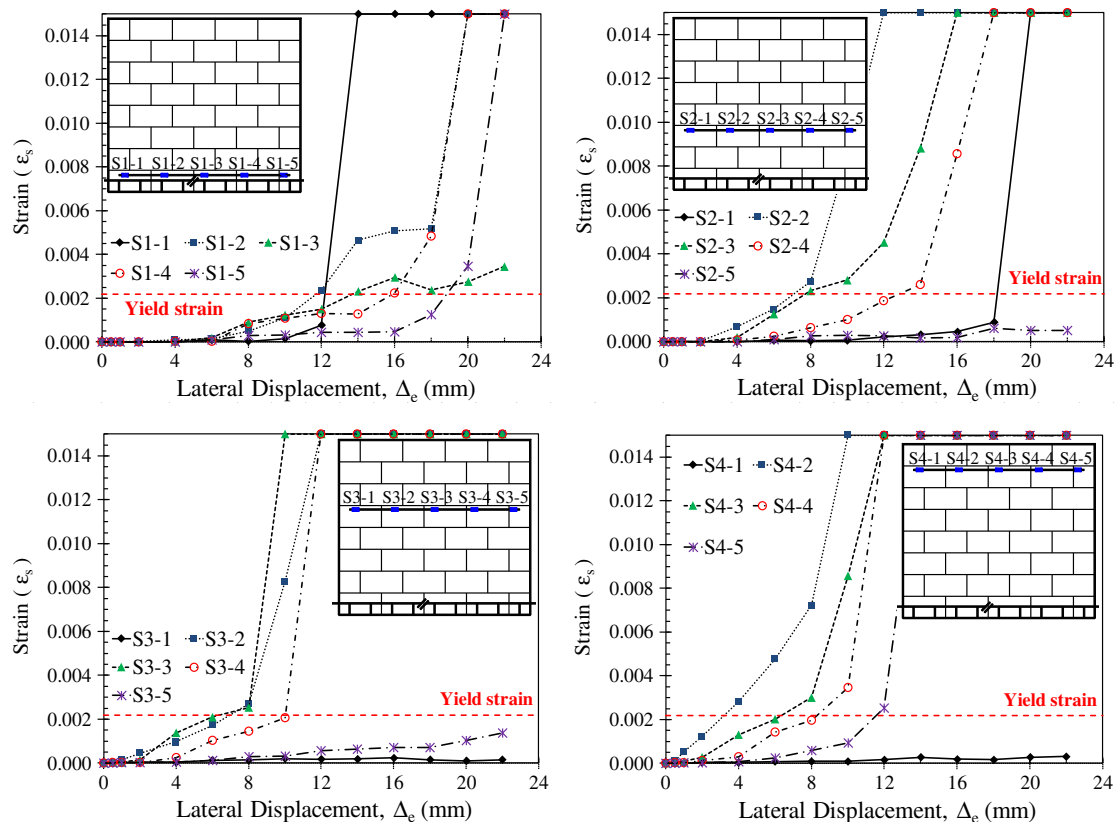
^aFirst major diagonal crack.

achieved lateral-load capacities higher than the calculated capacities presented in Table 1 with displacement ductility values higher than 3. As the figure and table show, the five walls had a similar behavior in both loading directions with a general symmetric resistance. Thus, only the results in the push direction were considered for the evaluation of the studied parameters. The experimental measurements for strength, displacements, and ductility are defined in the Notation list.

Analysis of Test Results for Wall W-Ref

To calculate the in-plane shear resistance provided by the horizontal reinforcement, V_s , the axial strain distribution along the horizontal

reinforcing bars was measured using five 5-mm strain gauges that were installed and equally distributed along the total length of each horizontal bar (Fig. 6). The strain gauges had a capacity of 15,000 microstrains, $\mu\epsilon$, which is less than the measured strain hardening. Hence, the axial tensile stress in each bar was limited to its yield capacity, f_y . Using the experimentally measured stress-strain relationship of reinforcement, Fig. 7 shows the stress distribution of the horizontal reinforcing bars at different levels of lateral excitations. Consequently, the average tension stress, f_s , in each bar was calculated while considering the crack pattern as follows: for Bar 1 (average of f1-1, f1-2, and f1-3); for Bar 2 (average of f2-2, f2-3, and f2-4); for Bar 3 (average of f3-2, f3-3, and f3-4); and for Bar 4 (average of f4-2, f4-3, and f4-4). After testing, the face shell was demolished to see the horizontal reinforcement. Fig. 8

**Fig. 6.** Lateral strain-displacement envelopes of horizontal reinforcing bars for Wall W-Ref

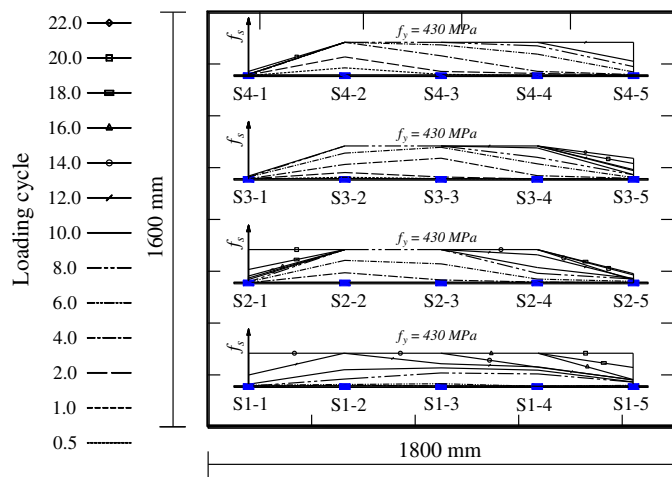


Fig. 7. Stress distribution of horizontal reinforcing bars for Wall W-Ref

presents the losses in the cross-sectional area of the horizontal reinforcing bars for Wall W-Ref after failure due to yielding. The tension forces in the horizontal reinforcing bars were then summed to calculate V_s . By subtracting the measured V_s from the measured in-plane lateral force, Q_e , the masonry and axial compressive stress resistance, V_{m+p} , was calculated.

The in-plane shear resistance contributions provided by masonry and axial compressive stress, V_{m+p} , and horizontal reinforcement, V_s , for Wall W-Ref at different levels of experimental displacement ductility, $\mu\Delta_e$, and top drift are presented in Fig. 9. The horizontal reinforcement reached its yield capacity at high displacement ductility and lateral top drift of 3.0 and 0.88%, respectively; however, at low deformations levels, this contribution was minor. Also, the rate of increase in V_s became higher after the initiation of diagonal cracks, which means that as the cracks increased and became wider, the horizontal reinforcement provided more resistance, V_s . As shown in the same figure, the in-plane lateral resistance of Wall W-Ref at low deformation levels is mainly provided by the masonry and axial compression force, V_{m+p} ; meanwhile, V_s was negligible. By increasing the level of deformations, the masonry reached its tensile strength capacity; hence, minor diagonal cracks occurred. These cracks decreased the rate of V_{m+p} contribution; on the other hand, it resulted in higher resistance provided by the horizontal reinforcement, V_s . As the cracks increased and became wider, the contribution rate of V_{m+p} decreased until the masonry reached its capacity, $(V_{m+p})_{\max}$. This is followed by a degradation in the resistance provided by the

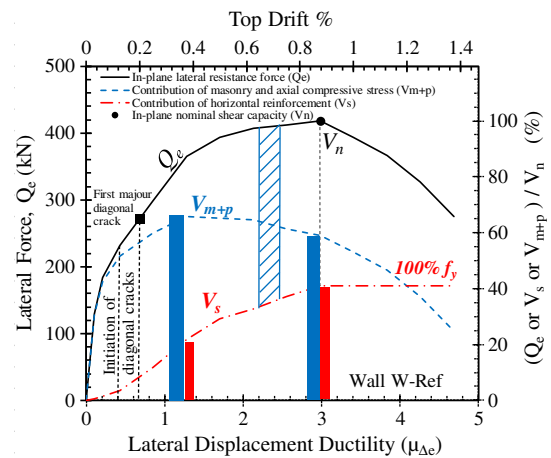


Fig. 9. In-plane shear resistance provided by masonry and axial compressive stress, V_{m+p} , and horizontal reinforcement, V_s , for Wall W-Ref

masonry and axial compressive stress. However, as long as the increase in V_s was higher than the reduction in V_{m+p} , the tested wall was able to provide more resistance.

Wall W-Ref reached its nominal in-plane shear capacity, V_n , when the horizontal reinforcement achieved its yield strength; then, degradation in the wall resistance occurred at the same rate of reduction as V_{m+p} . From Fig. 9, it can be noticed that the resistance shares provided by the masonry and axial compressive stress, V_{m+p} , and the horizontal reinforcement, V_s , did not reach their capacities at the same level of top drift. The resistance provided by the masonry and axial compressive stress, V_{m+p} , achieved its maximum contribution of $0.66V_n$ at displacement ductility, $\mu\Delta$, equal to 1.27, while V_s at the same $\mu\Delta = 1.27$ was $0.22V_n$. However, the horizontal reinforcement resistance, V_s , reached its yield capacity at $\mu\Delta$ of 3.0 with a contribution of $0.41V_n$ when the corresponding V_{m+p} reduced gradually to $0.59V_n$. It was observed that after the tested wall reached its capacity, V_n , the degradation in V_{m+p} became more rapid.

Fig. 10 presents the final crack pattern for Wall W-Ref after the first cycle of imposed top lateral displacement, Δ , equal to +22.0 mm. Minor shear cracks were observed at the cycle of ± 2.00 mm. However, the first sign of diagonal cracking damage occurred at lateral top displacement of +3.2 mm during the cycle of +4.0 mm. This first damage consisted of six diagonal cracks in the top left diagonal part of the tested wall with varied angles. By increasing the imposed lateral top displacement, the cracks

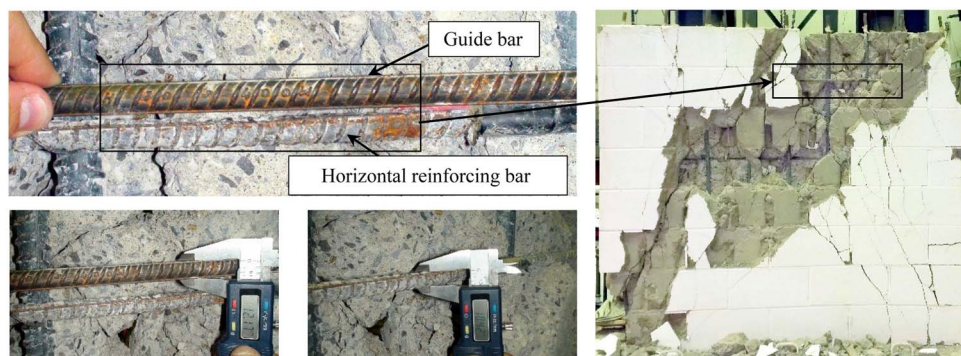


Fig. 8. Yielding of horizontal reinforcing bars for Wall W-Ref

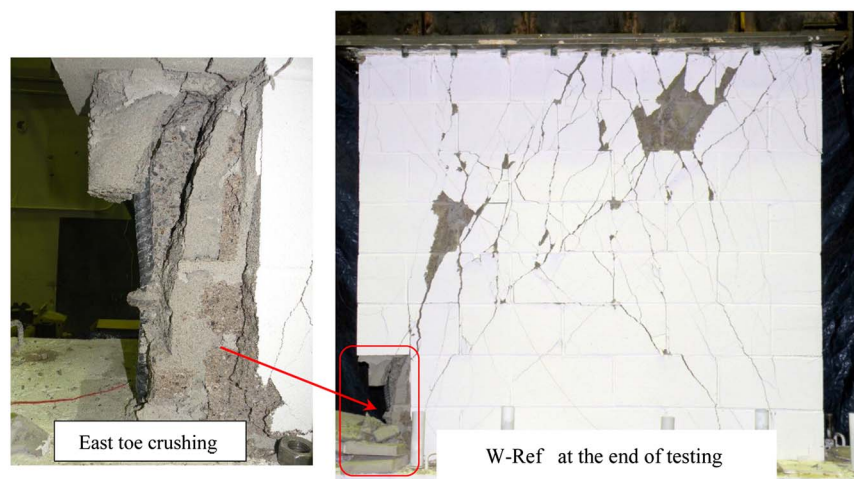


Fig. 10. Final crack pattern of tested Wall W-Ref

extended, increased, and gradually spread over the wall diagonals. Through the cycle of ± 6.00 mm, initial horizontal cracks appeared in the bed joint of the extreme cells from both sides in the first and second course from the base. Cracks continued to form in this manner until the tested wall reached its peak lateral load. During the postpeak behavior, the cracks were significantly wider and the face shell started to spall in addition to crushing in the east toe.

Effect of Horizontal Reinforcement Ratio

The effect of horizontal reinforcement in enhancing the in-plane shear behavior of RM walls in terms of shear strength and displacement ductility is given in Fig. 11. Providing a horizontal reinforcement ratio (ρ_h) of 0.13% increased the shear capacity from 365 kN. In addition, the tested Wall W-Ref was able to achieve a higher displacement ductility, $\mu_{\Delta 0.8Q_{ue}}$, of 4.2 compared to 3.0 for Wall W- $\rho_h 0$. However, it is clear from the test results that the horizontal reinforcement does not have a significant effect on the precrack behavior and it has a minor effect on the yield capacity since it increased from 309 kN for Wall W- $\rho_h 0$ to 328 kN for Wall W-Ref with just a 6% increase. This minor effect could be attributed to the buckling restraining effect of the 180° hooked horizontal reinforcement.

The experimentally measured resistance shares provided by masonry and axial compressive stress, W-Ref - V_{m+p} , and horizontal reinforcement, W-Ref - V_s , of Wall W-Ref against the lateral

resistance of Wall W- $\rho_h 0$, which is provided only by masonry and axial compressive stress and can be written as W- $\rho_h 0$ - V_{m+p} , are presented in Fig. 11. As shown in this figure, when Wall W- $\rho_h 0$ was constructed without horizontal reinforcement, W- $\rho_h 0$ - V_{m+p} reached 365 kN at displacement ductility of 2.1 followed by a rapid degradation in the strength to 292 kN, $0.8Q_{ue}$, at μ_{Δ} equal to 3.0. On the other hand, the inclusion of horizontal reinforcement in Wall W-Ref resulted in redistribution of the resistance provided by W-Ref - V_{m+p} . The shear resistance contribution provided by masonry and axial compressive stress, V_{m+p} , reached its ultimate lateral resistance of 275 kN at μ_{Δ} equal to 1.28. Although $(V_{m+p})_{\max}$ for W-Ref is 25% less than $(V_{m+p})_{\max}$ for wall W- $\rho_h 0$, Fig. 11 shows that the contribution of V_{m+p} became more ductile in Wall W-Ref compared to Wall W- $\rho_h 0$.

Most of the existing design equations for in-plane shear strength of RM shear walls, V_n , only consider a percentage of the horizontal reinforcement yield capacity contributing to V_n . The reason for this is that previous researchers did not quantify the resistance provided by the horizontal reinforcement, V_s , directly, but rather quantified V_s based on the difference between the achieved maximum lateral loads for tested RM walls with different horizontal reinforcement ratios. This quantification approach overlooks the effect of the redistribution in the shear resistance shares between the reinforcement and masonry, especially at high levels of displacement ductility. The presented test results here support this explanation, since the quantified resistance provided by the horizontal reinforcement reached 100% of its yield capacity. However, if the peak lateral load, Q_{ue} , of Walls W-Ref and W- $\rho_h 0$ was compared, it would falsely appear that the resistance provided by the horizontal reinforcement is 53 kN, which is only 30% of its yield capacity.

Fig. 12 shows the crack pattern of the two tested walls at different levels of deformation. The first sign of cracking for both Walls W- $\rho_h 0$ and W-Ref occurred at a similar top drift of 0.2%. This observation confirms that the horizontal reinforcement does not have a significant effect on precrack behavior. However, by increasing the imposed top lateral displacement, the main diagonal crack of Wall W- $\rho_h 0$ became wider. Distributing the horizontal reinforcement along the height of Wall W-Ref, replaced the major diagonal cracks with sets of minor cracks. These minor cracks were formed and gradually spread over the wall diagonals, as the initial diagonal cracks did not widen significantly. Moreover, the presence of the horizontal reinforcement delayed the corresponding top drift to the

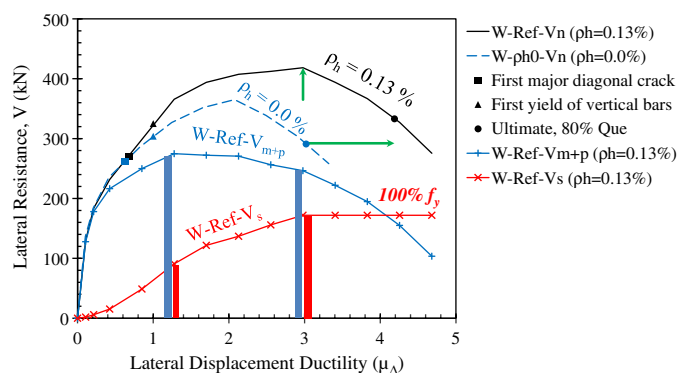


Fig. 11. Effect of horizontal reinforcement on in-plane shear strength of RM shear walls

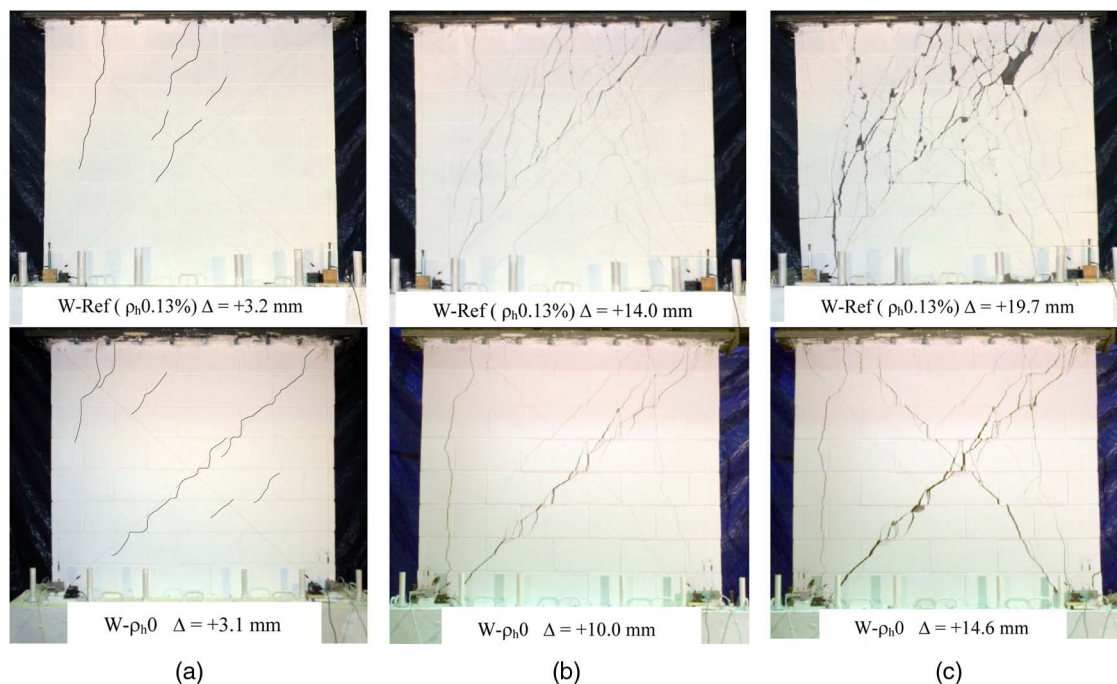


Fig. 12. Effect of horizontal reinforcement on crack pattern at (a) first major diagonal cracks; (b) lateral peak load Q_{ue} ; (c) when the lateral load dropped to 80% of Q_{ue}

maximum load, Q_{ue} , to 0.88% for Wall W-Ref instead of 0.63% for Wall W- ρ_h0 [Fig. 12(b)]. At a similar top drift of 0.91%, Wall W- ρ_h0 had a drop in its shear capacity to $0.8Q_{ue}$ with severe damage, whereas Wall W-Ref reached the failure displacement, $\Delta_{0.8Q_{ue}}$, at a top drift of 1.23% with major damage. Finally, the failure mode of Wall W- ρ_h0 can be classified as a brittle shear failure with two major diagonal cracks, while the behavior of Wall W-Ref is characterized by more-ductile performance with a higher displacement ductility [Fig. 12(c)].

Effect of Axial Compressive Stress

Increasing the axial compressive stress (σ_n) from 0.0 to 1.0 and 1.5 MPa resulted in a higher ultimate lateral force, Q_{ue} , from 345 to 418 and 458 kN, respectively, as shown in Fig. 13. Similar influence on diagonal crack load can be observed. The measured average yield displacement and the corresponding lateral load for Wall W- σ_n0 were 4.2 mm and 254 kN, respectively. These values increased to 4.7 mm and 328 kN for Wall W-Ref($\sigma_n1.0$) and 5.0 mm and 365 kN for wall W- $\sigma_n1.5$. However, this increase in the shear strength was followed by a reduction in the displacement ductility capacity, μ_{Δ_e} . At a top drift of 1.0%, $\mu_{\Delta_e1\%}$ decreased from 3.8 to 3.4 and 3.2 for Walls W- σ_n0 , W-Ref, and W- $\sigma_n1.5$, respectively. Moreover, the achieved ductility at failure, $\mu_{\Delta_{0.8Q_{ue}}}$, was measured as 3.4 for Wall W- $\sigma_n1.5$ compared to 4.1 for Wall W- σ_n0 .

The in-plane shear resistance provided by the horizontal reinforcement, V_s , reached its yield capacity for the three walls as presented in Fig. 14(a). The combined contribution of the masonry and axial stress, V_{m+p} , was calculated and is given in Fig. 14(b). The $(V_{m+p})_{\max}$ increased from 182 kN for Wall W- σ_n0 to 275 and 336 kN for Walls W-Ref and W- $\sigma_n1.5$, respectively. Since Wall W- σ_n0 was tested without axial compressive stress, the improvement in $(V_{m+p})_{\max}$ for Walls W-Ref and W- $\sigma_n1.5$ can be assumed as V_p , the resistance provided by the axial compressive stress only. Hence, V_p was calculated as $0.27P$ and $0.3P$ when σ_n was 1.0 and

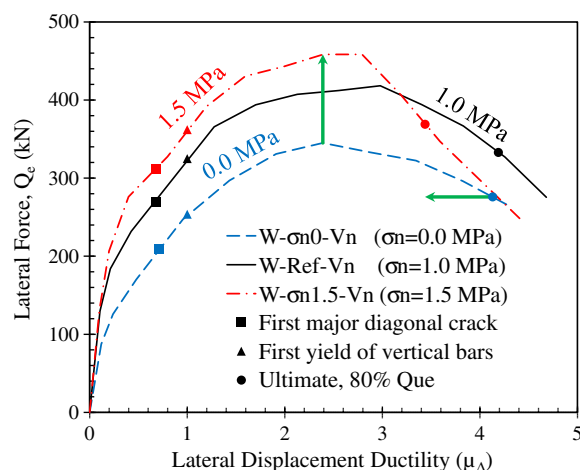


Fig. 13. Effect of axial compressive stress on in-plane shear strength of RM shear walls

1.5 MPa, respectively, where P is the equivalent compression axial load of σ_n . Nevertheless, comparing the ultimate lateral force resistances, Q_{ue} , of the three walls shows that V_p is only contributing 0.21 P and 0.23 P for Walls W- $\sigma_n1.0$ and W- $\sigma_n1.5$, respectively. These different values of V_p could be explained by delaying in the resistance provided by the horizontal reinforcement, V_s , as the axial compression load increased [Fig. 14(a)]. In addition, V_p reached its maximum contribution at early levels of displacement ductility, followed by a reduction in its resistance share as μ_{Δ_e} increased until the tested walls reached their capacity, V_n . Earlier researchers such as Sveinsson et al. (1985), Matsumura (1986), Shing et al. (1990), and Voon and Ingham (2006), evaluated the in-plane shear resistance provided by the axial compressive stress, σ_n , based on differences in the achieved ultimate lateral

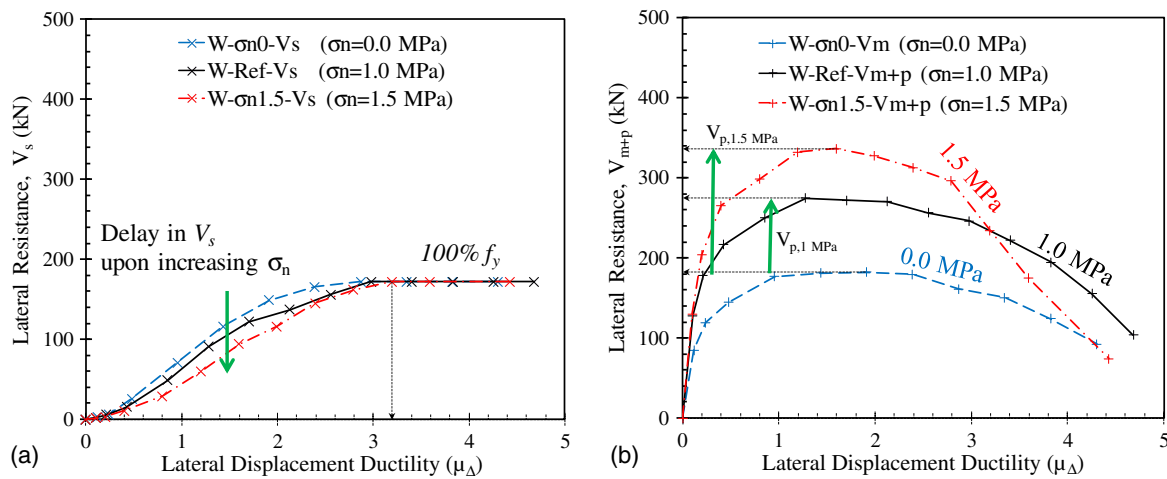


Fig. 14. Effect of axial compressive stress on shear resistance shares provided by (a) horizontal reinforcement, V_s ; (b) masonry and axial compressive stress, V_{m+p}

force, Q_{ue} , by testing RM walls under varying levels of σ_n . The test results presented here show that using this approach is underestimating V_p .

Fig. 15 shows the cracking patterns for Walls $W-\sigma_n 0$ and $W-\sigma_n 1.5$ at different stages: first major diagonal cracks, lateral peak load, Q_{ue} , and when the lateral load dropped to 80% of Q_{ue} . In general, the two walls exhibited a mixed shear-flexural behavior with a high contribution from the shear deformation towards the overall performance. Fig. 15(a) shows the first sign of cracking for the tested walls. The crack pattern in Wall $W-\sigma_n 0$ shows the ability of the test setup to distribute the applied lateral load along the length of the tested wall. Furthermore, increasing the axial compressive stress, σ_n , resulted in a higher angle between the diagonal shear cracks and the bed joint plane, θ , followed by less intensity of the initiated cracks. These observations support that increasing σ_n

enhanced the initial stiffness of the tested walls. At a top drift of 0.63%, Wall $W-\sigma_n 0$ achieved its ultimate lateral load resistance with major damage; however, Wall $W-\sigma_n 1.5$ reached its capacity at higher top drift of 0.88% with less damage [Fig. 15(b)]. During the postpeak behavior, Wall $W-\sigma_n 1.5$ had less damage compared to Walls $W-Ref$ and $W-\sigma_n 0$. This could be attributed to the enhancement of the aggregate interlocking at high levels of compressive stress. However, the absence of σ_n in Wall $W-\sigma_n 0$ increased the sliding deformation at the top course at late loading stages close to the failure displacement, as shown in Fig. 15(c). From the performance of three tested Walls, $W-\sigma_n 0$, $W-Ref(\sigma_n 1.0)$, and $W-\sigma_n 1.5$, it could be concluded that although the shear failure is mostly expected at the base where the maximum loads are, sliding shear failure could occur at the top of a RM shear wall in a multistory building since the gravity loads at the top are less than

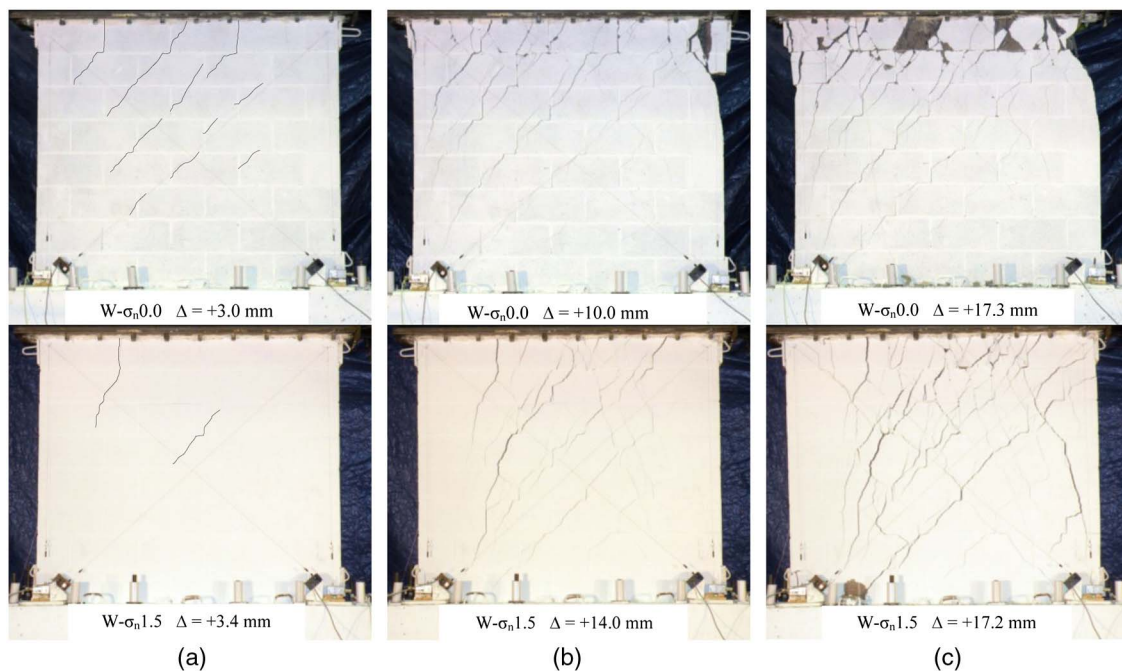


Fig. 15. Effect of axial compressive stress on crack pattern at (a) first major diagonal cracks; (b) lateral peak load Q_{ue} ; (c) when the lateral load dropped to 80% of Q_{ue}

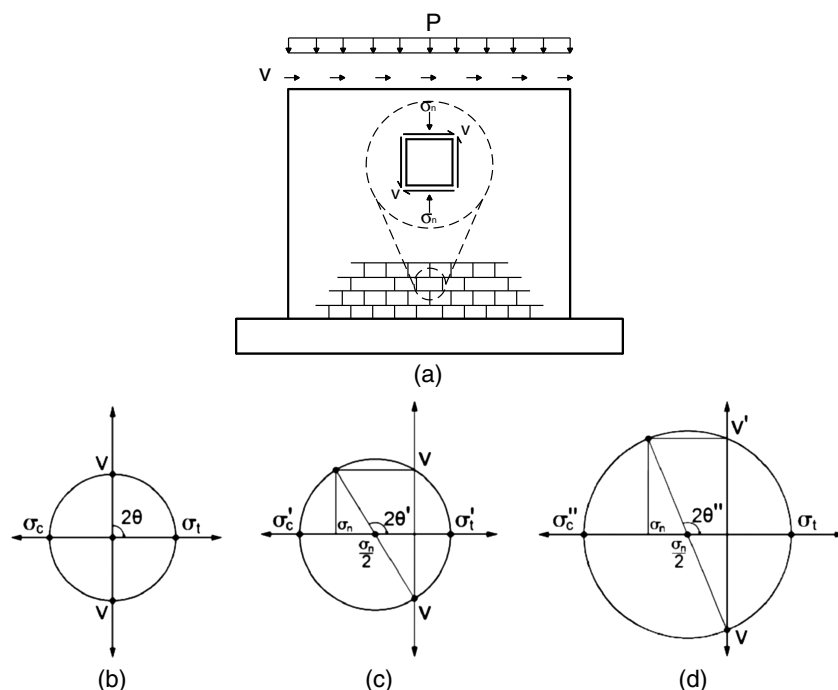


Fig. 16. Principal stresses acting on the masonry wall under different levels of axial compressive stress, σ_n

at the base. However, this type of failure is more common in partially grouted masonry walls without sufficient vertical reinforcement, i.e., dowel action.

The effect of axial compressive stress on the crack pattern can be explained by the principal stresses using Mohr's circle. Fig. 16 illustrates the principal stresses acting on a masonry panel where a two-dimensional state of stress develops in the wall: axial compressive stress, σ_n , and shear stress. The principal stresses due to uniform shear stress, v , and before applying σ_n are σ_t and σ_c . When the principal tensile stress, σ_t , exceeds the tensile strength of masonry, the initiation of diagonal shear cracks takes place in masonry walls. However, the presence of the axial compressive stress, σ_n , increases the principal compressive stresses to be σ'_c ; meanwhile, it reduces the principal tensile stresses to be σ'_t due to the compressive field created by σ_n , as shown in Fig. 16(c). Hence, to reach the same tensile strength, σ_t , a higher value of shear stress, v' , is required [Fig. 16(d)]. Moreover, it is clear from the same figure that increasing σ_n results in a higher angle of the principal stresses, θ' .

Effect of Shear Span to Depth Ratio

One of the important parameters that influence the inelastic behavior of RM shear walls is the shear span to depth ratio, M/Vd_v . Most of the existing design codes for masonry structures, including the Canadian Standards Association CSA S304 (CSA 2014b), the U.S. Masonry Standards Joint Committee (MSJC 2013), and New Zealand Standards NZS 4230 (NZS 2004), limit the effect of M/Vd_v to an upper limit of 1.0. Wall W-M/ Vd_v 1.8 was subjected to a top moment of $0.9V$ kN · m, where V is the lateral load from the horizontal actuator. Instead, Wall W-Ref was tested without a top moment. As proposed by the Canadian Standards Association CSA S304 (CSA 2014b), the effective shear depth, d_v , was taken as $0.8l_w$. Thus, the value of M/Vd_v for Walls W-Ref and W-M/ Vd_v 1.8 are 1.25 and 1.875, respectively. Since both walls are identical in dimensions and reinforcement, comparison between

their behaviors aims to evaluate the effect of M/Vd_v on the in-plane shear performance of RM shear walls.

The test results presented in Fig. 17 show a reduction in the achieved ultimate force, Q_{ue} , by 25% due to increasing M/Vd_v from 1.25 to 1.875. This loss in the shear resistance is accompanied by enhancement in the displacement ductility, $\mu_{\Delta 0.8Q_u}$, from 4.2 to 5.2. Moreover, the crack strength of Wall W-M/ Vd_v 1.8 was 30% less than Wall W-Ref. Wall W-M/ Vd_v 1.8 achieved its yield capacity at a lateral load, Q_{ye} , of 230 kN compared to 328 kN when the same wall was tested without a top moment. The effect of M/Vd_v on shear resistance shares provided by horizontal reinforcement, V_s , and masonry and axial compressive stress, V_{m+p} , is given in Fig. 18. The shear span to depth ratio does not have a significant effect on V_s since the horizontal reinforcement in both walls reached its yield capacity at the same μ_{Δ} of 3.0. At an early stage of ductility, the shear resistance provided by V_s in Wall

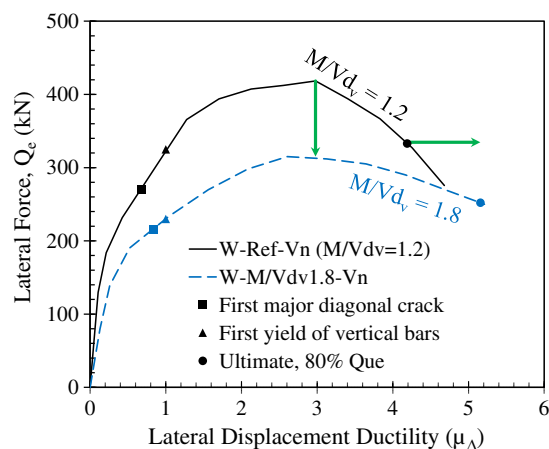


Fig. 17. Effect of shear span to depth ratio, M/Vd_v , on in-plane shear strength of RM shear walls

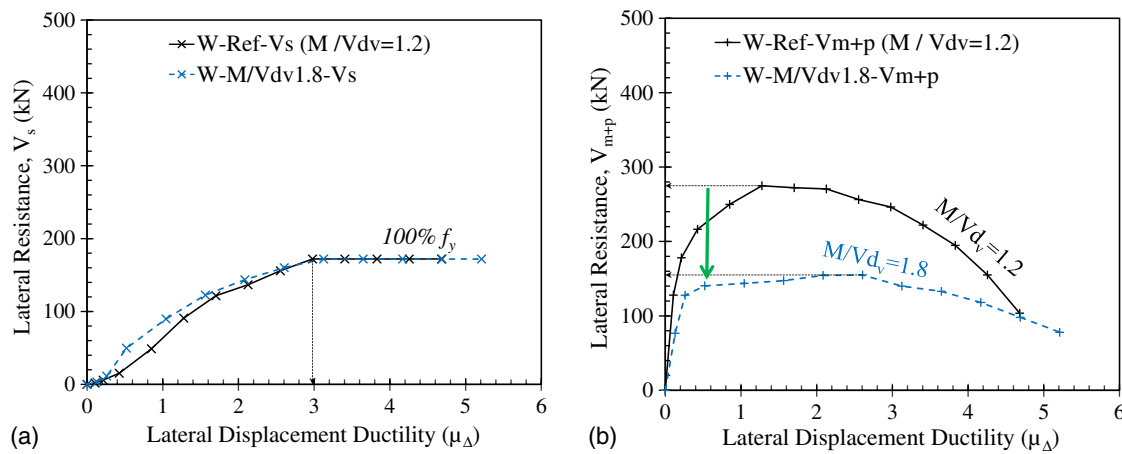


Fig. 18. Effect of shear span to depth ratio, M/Vd_v , on shear resistance shares provided by (a) horizontal reinforcement, V_s ; (b) masonry and axial compressive stress, V_{m+p}

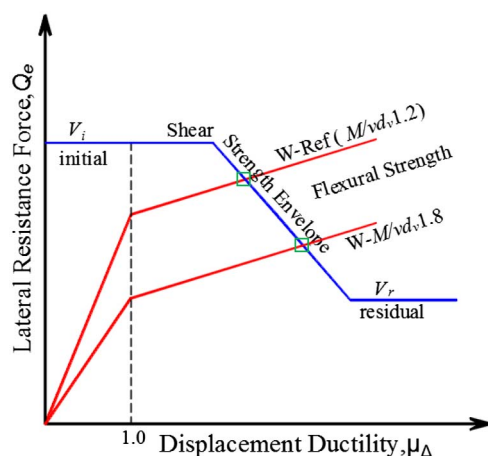


Fig. 19. Interaction between flexural and shear performance of Walls W-Ref and W-M/ Vd_v 1.8

W-M/ Vd_v 1.8 was higher than W-Ref because Wall W-M/ Vd_v 1.8 reached its inelastic deformations earlier. On the other hand, the aforementioned reduction in Q_{ue} due to increasing M/Vd_v from 1.25 to 1.875, could be due to the losses in $(V_{m+p})_{max}$ as shown in Fig. 18(b). In Wall W-Ref, the masonry and axial compressive stress contributed with an ultimate resistance of 275 kN at μ_Δ of 1.27, followed by rapid strength degradation. This behavior became more ductile in Wall W-M/ Vd_v 1.8 with less capacity, $(V_{m+p})_{max}$, equal to 155 kN.

The test results of Walls W-Ref and W-M/ Vd_v 1.8 show that limiting the effect of shear span to depth ratio to an upper value of 1.0, as provided in most masonry design codes, is overestimating the in-plane shear strength of RM shear walls, V_n , at high values of M/Vd_v , which might lead to an unsafe design. This experimentally measured reduction in the shear capacity could be explained by considering the interaction between the flexural and shear performances, as shown in Fig. 19. Both walls are identical in dimensions and reinforcement. Hence, they would theoretically have the same shear strength envelope and moment capacity according to Canadian Standards CSA S304 (CSA 2014b). Increasing M/Vd_v results in a reduction in the lateral force that corresponds to the flexural capacity of the wall. Consequently, this lower lateral force

intersects with the shear strength envelope at a lower capacity along with higher displacement ductility.

Both Walls W-Ref and W-M/ Vd_v 1.8 had similar crack propagation and were characterized by moderately ductile failure as shown in Fig. 20. Initial diagonal crack damage was observed at a top drift of 0.2% for both walls. As the imposed in-plane top lateral displacement increased, more cracks were formed and gradually spread over the wall diagonals. Wall W-M/ Vd_v 1.8 reached its lateral load capacity at early levels of deformation when subjected to top displacement of +10.0 mm; in contrast, Wall W-Ref was able to gain more resistance until Δ_e equal to +14.0 mm. However, Wall W-M/ Vd_v 1.8 required more loading cycles to lose 20% of its shear strength, Q_{ue} , compared to Wall W-Ref. The two tested walls reached their failure points at almost the same Δ_e with an average top drift of 1.23%. However, increasing M/Vd_v from 1.25 to 1.875 resulted in higher levels of axial compressive stress on the end zones of Wall W-M/ Vd_v 1.8. Hence, failure occurred in the wall cover as presented in Fig. 21. As shown in this figure, the right side view of the final crack pattern for Wall W-M/ Vd_v 1.8 does not display any internal crushing in the grout, which could explain the enhancement in the stiffness degradation during the postpeak behavior.

Summary and Conclusions

This paper assessed experimentally the in-plane shear behavior of five single-story RM shear walls when subjected to in-plane axial compressive stress, cyclic lateral excitations, and top moment. The studied parameters were the horizontal reinforcement ratio, level of axial compressive stress, and shear span to depth ratio. The test results support the following conclusions for fully grouted rectangular RM shear walls:

- Providing horizontal reinforcement to masonry shear walls enhances their in-plane shear behavior in terms of shear strength and displacement ductility. However, the horizontal reinforcement does not have a significant influence on the precrack behavior and it has a minor effect on the yield capacity. On the other hand, the experimentally measured in-plane shear resistance provided by the horizontal reinforcement, V_s , reached 100% of its yield capacity for all tested walls at high displacement ductility. These results do not agree with most of the existing design equations for in-plane shear strength of RM shear

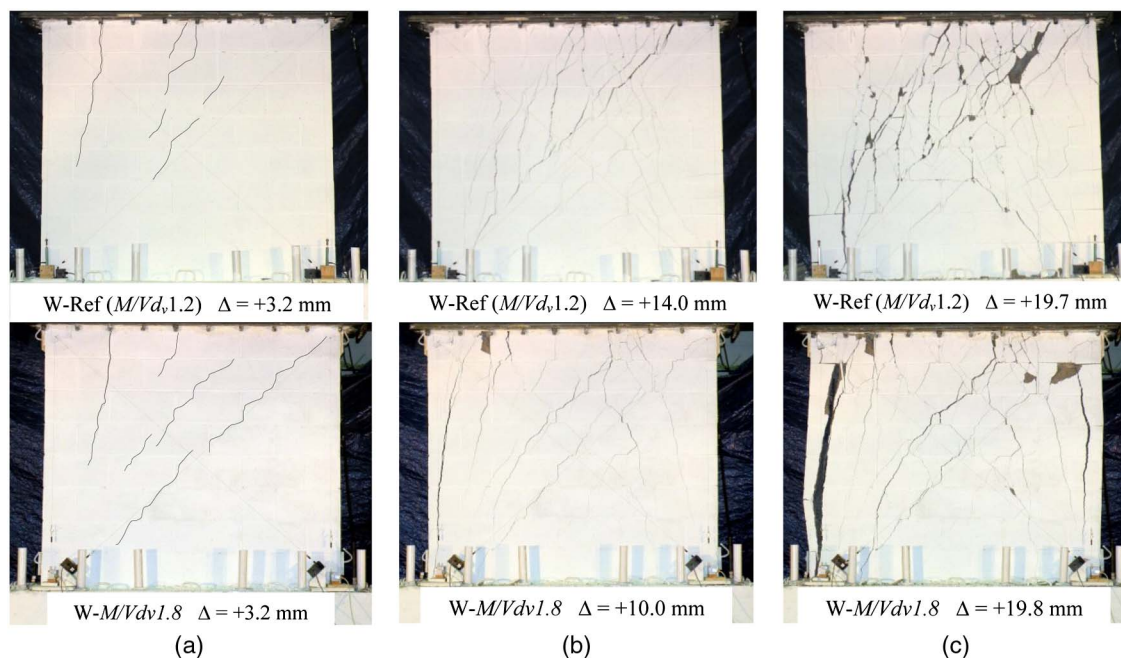


Fig. 20. Effect of shear span to depth ratio, M/Vd_v , on crack pattern at (a) first major diagonal cracks; (b) lateral peak load Q_{ue} ; (c) when the lateral load dropped to 80% of Q_{ue}



Fig. 21. Final crack pattern of tested Wall W-M/ Vd_v 1.8

walls, V_n , that consider only a percentage of the yield capacity of the horizontal reinforcement contributing to V_n ;

- Increasing the axial compressive stress, σ_n , results in a higher shear strength accompanied with more brittle failure. Likewise, it enhances the initial stiffness but it is followed by rapid stiffness degradation. From the performance of tested walls under different levels of σ_n , it can be concluded that although the shear failure is mostly expected at the base where the maximum loads are, sliding shear failure could occur at the top of a RM shear wall in a multistory building since the gravity loads at the top are less than at the base. However, this type of failure is more common in partially grouted masonry walls without sufficient vertical reinforcement, i.e., limited dowel action;
- The test results of walls W-Ref and W-M/ Vd_v 1.8 with shear span to depth ratio, M/Vd_v equal to 1.25 and 1.875, respectively, show

a significant reduction in the shear strength by 25% when M/Vd_v is increased. These results clarify that limiting the effect of M/Vd_v to an upper value of 1.0, as provided in most of the masonry design codes, is overestimating V_n of RM shear walls at high values of M/Vd_v , which might lead to an unsafe design. However, W-M/ Vd_v 1.8 was able to achieve higher levels of displacement ductility. This behavior could be explained by considering the interaction between the flexural and shear performances.

Acknowledgments

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Notation

The following symbols are used in this paper:

- Q_{ue} = peak lateral force;
- Q_{ye} = lateral yield force, defined at the lateral yield displacement, Δ_{ye} , on the backbone curve;
- $\Delta_{Q_{ue}}$ = lateral displacement at the peak lateral force, Q_{ue} ;
- Δ_{ye} = lateral yield displacement, taken as the average between the top lateral displacements that are corresponding to the first yield in the vertical reinforcement in each direction;
- $\Delta_{0.8Q_{ue}}$ = lateral displacement defined at a drop in wall capacity to 80% of Q_{ue} ;
- $\mu_{\Delta_{e1\%}}$ = lateral displacements ductility corresponding to the top drift of 1.0% = $\Delta_{e1\%}/\Delta_{ye}$;
- $\mu_{\Delta_{Q_{ue}}}$ = lateral displacements ductility at peak force = $\Delta_{Q_{ue}}/\Delta_{ye}$; and
- $\mu_{\Delta_{0.8Q_{ue}}}$ = lateral displacements ductility at a drop in wall capacity to 80% of Q_{ue} = $\Delta_{0.8Q_{ue}}/\Delta_{ye}$.

References

- Anderson, D. L., and Priestley, M. J. N. (1992). "In plane shear strength of masonry walls." *Proc., 6th Canadian Masonry Symp.*, Univ. of Saskatchewan, Saskatoon, SK, Canada, 223–234.
- ASTM. (2010a). "Standard specification for grout for masonry." *ASTM C476-10*, West Conshohocken, PA.
- ASTM. (2010b). "Standard test methods for flexural bond strength of masonry." *ASTM E518-10*, West Conshohocken, PA.
- ASTM. (2011). "Standard test method for splitting tensile strength cylindrical concrete specimens." *ASTM C496/C496M-11*, West Conshohocken, PA.
- ASTM. (2012a). "Standard test method for compressive strength of cylindrical concrete specimens." *ASTM C39/C39M-12*, West Conshohocken, PA.
- ASTM. (2012b). "Standard test method for compressive strength of masonry prisms." *ASTM C1314-12*, West Conshohocken, PA.
- ASTM. (2013a). "Standard specification for deformed and plain carbon-steel bars for concrete reinforcement." *ASTM A615/A615M-13*, West Conshohocken, PA.
- ASTM. (2013b). "Standard test method for flow of hydraulic cement mortar." *ASTM C1437-13*, West Conshohocken, PA.
- ASTM. (2014). "Standard test methods and definitions for mechanical testing of steel products." *ASTM A370-14*, West Conshohocken, PA.
- CSA (Canadian Standards Association). (2004). "Mortar and grout for unit masonry." *CSA A179-04*, Mississauga, Canada.
- CSA (Canadian Standards Association). (2014a). "Concrete block masonry units." *CSA A165.1-14*, Mississauga, Canada.
- CSA (Canadian Standards Association). (2014b). "Design of masonry structures." *CSA S304-14*, Mississauga, Canada.
- El-Dakhkhni, W., Banting, B., and Miller, S. (2013). "Seismic performance parameter quantification of shear-critical reinforced concrete masonry squat walls." *J. Struct. Eng.*, [10.1061/\(ASCE\)ST.1943-541X.0000713](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000713), 957–973.
- FEMA. (2007). "Interim testing protocols for determining the seismic performance characteristics of structural and nonstructural components." *FEMA Rep. No. 461*, Applied Technology Council, Washington, DC.
- FEMA. (2009). "Damage states and fragility curves for reinforced masonry shear walls." *FEMA P-58/BD-3.8.10*, Applied Technology Council, Washington, DC.
- Haach, V., Vasconcelos, G., and Lourenço, P. (2010). "Experimental analysis of reinforced concrete block masonry walls subjected to in-plane cyclic loading." *J. Struct. Eng.*, [10.1061/\(ASCE\)ST.1943-541X.0000125](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000125), 452–462.
- Kaminosono, T., Teshigowara, M., Hiraishi, H., Fujisawa, M., and Nakaoka, A. (1988). "Experimental study on seismic performance of reinforced masonry walls." *Proc., 9th World Conf. on Earthquake Engineering*, Japan Association for Earthquake Disaster Prevention, Japan, 109–114.
- Matsumura, A. (1986). "Shear strength of reinforced hollow unit masonry walls." *2nd Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research*, U.S. Japan Cooperative Research Program, CO.
- MSJC (Masonry Standards Joint Committee). (2013). "Building code requirements for masonry structures." *TMS 402/ ACI 530/ASCE 5*, ASCE, Reston, VA.
- SANZ (Standards Association of New Zealand). (2004). *Design of reinforced concrete masonry structures*, NZS 4230, Wellington, New Zealand.
- Seif Eldin, H., and Galal, K. (2015). "Survey of design equations for the in-plane shear capacity of reinforced masonry shear walls." *Proc., 12th North American Masonry Conf.*, The Masonry Society (TMS), Longmont, CO.
- Shing, P., Schuller, M., and Hoskere, V. (1990). "In-plane resistance of reinforced masonry shear walls." *J. Struct. Eng.*, [10.1061/\(ASCE\)0733-9445\(1990\)116:3\(619\)](https://doi.org/10.1061/(ASCE)0733-9445(1990)116:3(619)), 619–640.
- Sveinsson, B. I., Mayes, R. L., and McNiven, H. D. (1985). "Cyclic loading of masonry single piers—Volume 4: Additional tests with height to width ratio of 1." *Rep. No. UCB/EERC-85/15*, Earthquake Engineering Research Centre, Univ. of California, Berkeley, CA.
- Tomažević, M., Lutman, M., and Petković, L. (1996). "Seismic behavior of masonry walls: Experimental simulation." *J. Struct. Eng.*, [10.1061/\(ASCE\)0733-9445\(1996\)122:9\(1040\)](https://doi.org/10.1061/(ASCE)0733-9445(1996)122:9(1040)), 1040–1047.
- Voon, K., and Ingham, J. (2006). "Experimental in-plane shear strength investigation of reinforced concrete masonry walls." *J. Struct. Eng.*, [10.1061/\(ASCE\)0733-9445\(2006\)132:3\(400\)](https://doi.org/10.1061/(ASCE)0733-9445(2006)132:3(400)), 400–408.