

Effect of reinforcement anchorage end detail and spacing on seismic performance of masonry shear walls

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ABSTRACT

The most recent design codes for masonry structures necessitate the use of reinforced masonry (RM) shear walls in medium and high seismic areas. In addition, they provide requirements for the seismic reinforcement for walls. This paper investigates the in-plane seismic performance of fully grouted RM shear walls dominated by shear-flexural failure. The experimental work involved assessing the response of five single-story RM shear walls when subjected to in-plane axial compressive stress and cyclic lateral excitations. The studied parameters were the horizontal reinforcement anchorage end detail and the spacing of the horizontal and vertical reinforcement. Three anchorage end detail were evaluated in this study; 180° standard hook, 90° hook, and straight. Two vertical spacing of the horizontal reinforcement, 400 mm and 800 mm, and two horizontal spacing of the vertical reinforcement, 200 mm and 800 mm, were considered in the test matrix. Based on the test results, the 180° standard hook was the most efficient in terms of strength and ductility with a slight difference in the strength and 15% higher ductility than using the straight end detail. On the contrary, the spacing of the reinforcement had a significant effect on the behavior of tested walls. Walls that were constructed with closely spaced reinforcement were able to reach 15% and 80% higher strength and ductility than similar walls with large spacing when using the same reinforcement ratio. Hence, the current values for the maximum spacing of reinforcement in the Canadian Standards Association CSA S304-14 for the design of masonry structures need to be modified by specifying lower spacing limits.

1. Introduction

The poor performance of unreinforced masonry buildings during post-earthquake reconnaissance has led to the development of reinforced masonry (RM) systems. Similar to reinforced concrete buildings, RM shear walls are the key structural elements widely used to resist lateral loads in masonry buildings due to their capability to provide lateral strength, stiffness, and energy dissipation. Most of the design equations for in-plane shear strength, V_n , neglect the effect of the spacing of the vertical and horizontal reinforcement and also, the anchorage end detail of the horizontal reinforcement [1]; instead, they consider the reinforcement ratios. However, to account for these effects, the recent masonry design codes provide some provisions for seismic reinforcement.

Many experimental studies have been conducted to investigate the effect of several parameters on the seismic performance of RM shear walls [2–9]. Some of these experimental works studied the influence of the spacing of reinforcement on the seismic performance of RM shear walls and their test results concluded that the spacing of reinforcement

has a considerable impact on their strength and ductility. Voon and Ingham [8] tested ten reinforced masonry walls under cyclic horizontal loading to evaluate their in-plane shear resistance. Out of these walls, two walls were constructed with approximately the same horizontal reinforcement ratio, ρ_h (0.05% and 0.06%), but with different distribution. First RM wall was constructed with $5 \times R6$ ($\rho_h = 0.05\%$) horizontal bars while the second wall had $2 \times D10$ ($\rho_h = 0.06\%$). Their test results showed that the wall with small spacing exhibited a more gradual strength degradation than the one with larger spacing. Similar experimental investigation was carried out by Shing et al. [9] and reported similar observation. Furthermore, there are few studies that have considered the effect of the horizontal reinforcement anchorage end detail on the in-plane shear behavior of RM walls, one of which is the experimental work conducted by [10] in which 90° bent, 180° hook, and end plate anchorages were investigated. Based on their results, for the walls with heavy lateral reinforcement, the end plate was the most effective anchorage end detail, whereas the 90° bent was the least effective. However, where the horizontal reinforcement was light, both the 180° hook and the end plate were more effective than the 90° bent

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and showed similar performance. Still, it is important to mention that all three types of anchorage end details enhanced the pre-diagonal crack behavior of the RM walls that were dominated by shear failure in terms of strength and ductility. Although these studies highlighted the effect of the distribution and the anchorage end detail of the horizontal reinforcement on the in-plane behavior of RM shear walls, there is a need for more experimental studies that quantify these effects.

This paper aims to quantify experimentally the effect of the horizontal reinforcement anchorage end detail and the spacing of reinforcement on the in-plane seismic performance of fully grouted RM shear walls dominated by shear-flexural failure. The test results discuss the shear resistance shares provided by horizontal reinforcement, V_s , and masonry and axial compressive stress, V_{m+p} ; the crack pattern; the stiffness degradation; and energy dissipation. Moreover, a brief discussion on the requirements for seismic reinforcement according to the Canadian standard CSA S304-14 [11] is presented, followed by recommendations for the upcoming edition.

2. Experimental work

2.1. Test matrix

The experimental work assesses the response of five full-scale fully grouted rectangular RM shear walls when subjected to in-plane axial compressive stress and cyclic lateral excitations. The main variables considered were the horizontal reinforcement anchorage end detail, the spacing of horizontal reinforcement, and the spacing of vertical reinforcement. Table 1 summarizes the reinforcement details of the tested walls, in addition to the theoretical lateral forces corresponding to the in-plane shear, flexural and sliding capacities according to the Canadian Standards CSA S304-14 [11]. As shown in this table, all the walls should have theoretically the same failure load. All the tested reinforced masonry walls had the same dimensions, $1.8 \text{ m} \times 1.6 \text{ m} \times 0.19 \text{ m}$, and were constructed with a constant horizontal reinforcement ratio of 0.13%. Each wall was constructed on a reinforced concrete (RC) foundation with dimensions of $2.3 \text{ m} \times 0.64 \text{ m} \times 0.45 \text{ m}$ (see Fig. 1). The walls were subjected to cyclic lateral excitations at a height of 1.80 m from the top of the RC foundation in order to keep the shear span to depth ratio higher than 1.0 and were tested under a constant axial compressive stress of 1.0 MPa.

All the loads were transferred to the tested walls through a stiff built-up steel loading beam. The vertical reinforcement of the RM walls was anchored to the bottom flange of the loading beam. The first wall, W-Ref, was designed to be a reference wall for the remaining tested walls. Wall W-Ref had uniformly distributed horizontal reinforcing

Table 1
Test matrix of five RM shear walls.

Wall ID	Reinforcement		Studied parameter	Theoretical capacities according to CSA S304-14				
	Vertical	Horizontal	Horizontal reinf. end detail	S_v^a	S_h^a	F_{Vn}^b	F_{Mn}^c	F_{Sn}^d
			–	mm	mm	kN	kN	kN
W-Ref	20 M@200	10 M@400	180° hook	400	200	269	541	1469
W-90°	20 M@200	10 M@400	90° hook	400	200	269	541	1469
W-Str	20 M@200	10 M@400	Straight	400	200	269	541	1469
W-S _v 800	20 M@200	15 M@800	180° hook	800	200	269	541	1469
W-S _h 800	30 M@800	15 M@800	180° hook	800	800	269	462	1211

^a S_v and S_h are the vertical and horizontal spacing of reinforcing, respectively, as shown in Fig. 1.

^b Theoretical in-plane shear capacity assuming moderately ductile shear walls ($R_d = 2.0$).

^c Theoretical lateral force corresponding to the flexural capacity assuming plane sections assumption.

^d Theoretical lateral force corresponding to the sliding capacity.

bars, 10 M@400 mm, with a standard 180° hook around the outermost vertical bars. The anchorage end detail of the horizontal reinforcement for walls W-90° and W-Str were 90° hook and straight bar, respectively. The standard 180° hook and 90° hook were detailed according to CSA S304-14 [11], where the hook tail length is bigger than $4d_b$ (45 mm) and $12d_b$ (135 mm) for the 180° hook and 90° hook, respectively, as shown in Fig. 2.

The impact of the horizontal reinforcement anchorage end detail on the seismic response of RM shear walls was assessed based on the test results of walls W-Ref, W-90°, and W-Str. Unlike wall W-Ref, wall W-S_v800 was constructed with horizontal reinforcement of 15 M@800 mm, in order to study the effect of vertical spacing between horizontal bars when using the same horizontal reinforcement ratio. Wall W-S_h800 is a duplicate of wall W-S_v800, except that the vertical reinforcement was concentrated in the first, middle, and end cells. Each cell has a 30 M bar with a total vertical reinforcement ratio, ρ_v , of 0.61% compared to ρ_v of 0.79% for the rest of the tested walls. However, this slight difference in ρ_v can be neglected since most of the existing equations for predicting the nominal in-plane shear strength, V_n , including the design equations given in the Canadian standard CSA S304-14 [11] and the Masonry Standards Joint Committee MSJC-2013 [12], do not consider the contribution of the vertical reinforcement.

2.2. Properties of materials

All the tested walls and the required auxiliary specimens were designed in accordance with the Canadian Standards CSA S304-14 [11] and CSA A179-14 [13] and were constructed by certified masons using full-scale lightweight knock-out concrete masonry units (CMUs) with nominal dimensions of $390 \text{ mm} \times 190 \text{ mm} \times 190 \text{ mm}$. Before the construction of the tested walls, the knockout webs were removed to accommodate the horizontal reinforcement and to provide grouting continuity in the vertical and horizontal directions, consequently, preventing any weakness planes between the concrete masonry units (see Fig. 3). The blocks were joined together with 10 mm type S mortar joints and laid in a running bond pattern, then, all the cells were fully grouted using coarse grout that was mixed in the laboratory. The nominal compressive strength of the block, mortar, and grout were measured experimentally and the average values were found to be 16.7 (c.o.v. = 4.8%), 13.7 (c.o.v. = 7.8%), and 29.4 MPa (c.o.v. = 7.3%), respectively. Masonry prisms were built at the completion of laying each wall in order to measure the masonry compressive strength for the tested walls. Four concrete masonry units (CMUs) were stacked on top of each other using the same construction materials that were used to build the walls. The average masonry compressive strength was found to be 13.1 MPa (c.o.v. = 7.6%). The average experimental measured yield strength of the steel reinforcement was 430 MPa (c.o.v. = 3.2%).

2.3. Test setup

The RM walls were tested under in-plane vertical and lateral loads using three MTS hydraulic actuators as shown in Fig. 4. Two actuators were installed vertically to apply the axial compression force and the horizontal actuator was used to introduce cyclic horizontal displacements to simulate the seismic loads. The applied loads were transferred to the tested walls through a stiff built-up steel-loading beam that was restrained to any out-of-plane displacement using two out-of-plane steel back-to-back angles that connected the loading steel beam (through slotted holes) to a strong RC wall. To provide a fixed boundary condition at the base of the tested walls, each wall was constructed on a reinforced concrete (RC) foundation that was connected to a strong floor using a reusable RC footing with a depth of 600 mm.

2.4. Instrumentation and testing protocol

Two different types of instruments were used to measure the strains

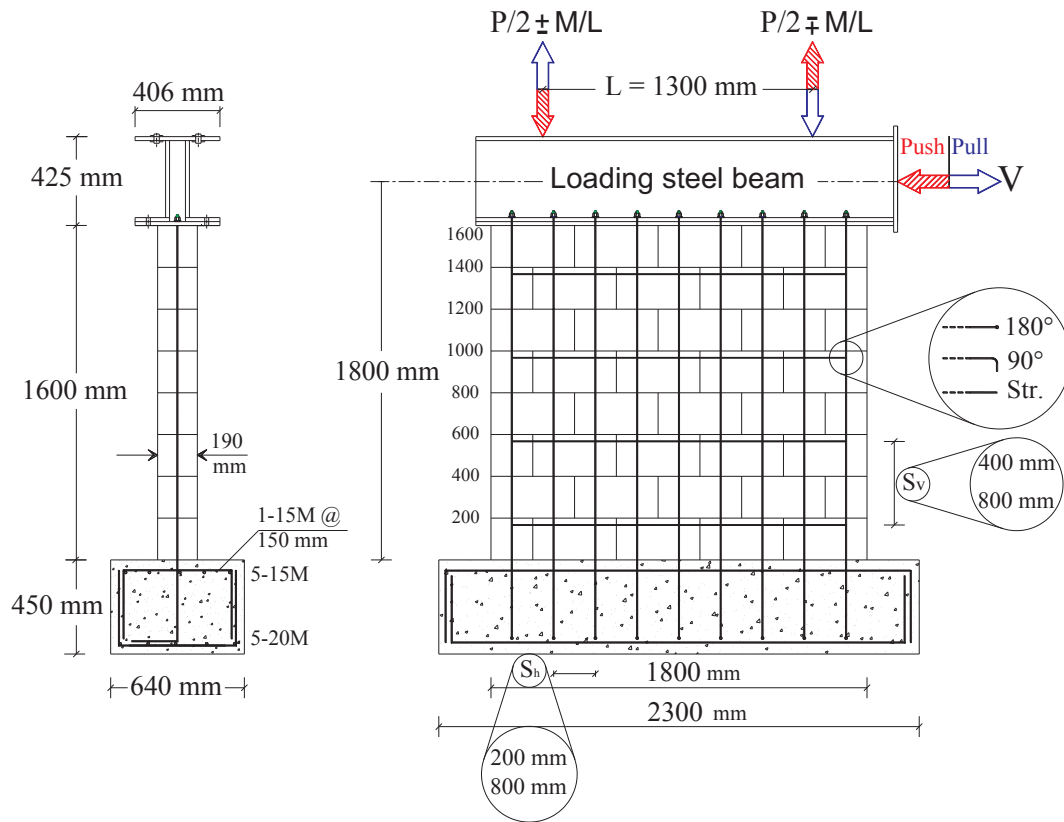
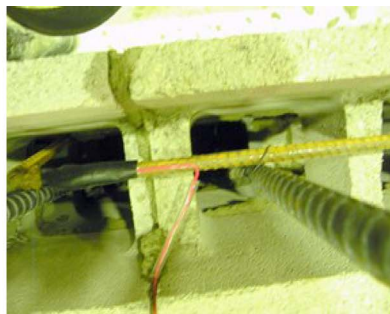


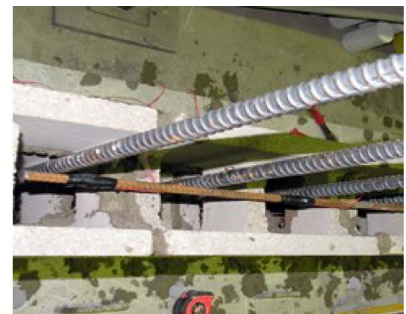
Fig. 1. Dimensions of a typical tested RM shear wall showing studied parameters.



(a) 180° hook



(b) 90° hook



(c) Str

Fig. 2. Horizontal reinforcement end details of tested RM shear walls: (a) W-Ref; (b) W-90°; (c) W-Str.

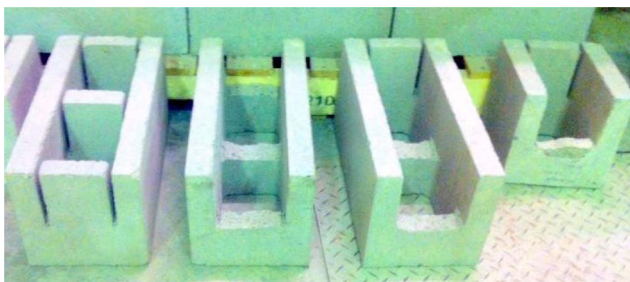


Fig. 3. Full-scale concrete masonry units (CMUs) with knockout webs.

in the reinforcement and to monitor the deformations of the tested walls during testing. Strain gauges were installed on the steel reinforcement bars to record the local strains during loading of the walls

as shown in Fig. 5-a. To define the yield load, four strain gauges were installed at the wall-footing interface of the two outermost vertical reinforcement bars on each side. Additionally, five strain gauges were installed and equally distributed along the total length of each horizontal bar to measure the axial strain distribution. Then, the average tension force in each bar was calculated and summed to find the horizontal reinforcement contribution, V_s , to the in-plane shear strength, V_n using the experimentally measured stress-strain relationship and the cross-section area for the horizontal steel reinforcing bars. The axial tensile stress in each bar was limited to its yield capacity, f_y , because of the strain gauges had a capacity of 15,000 microstrain, $\mu\epsilon$, which is less than the measured strain hardening. 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 [14]. In addition to the strain gauges, the horizontal, vertical, and diagonal displacements were measured using nine linear potentiometers that were attached externally to each of the tested RM shear walls as presented in Fig. 5-b.

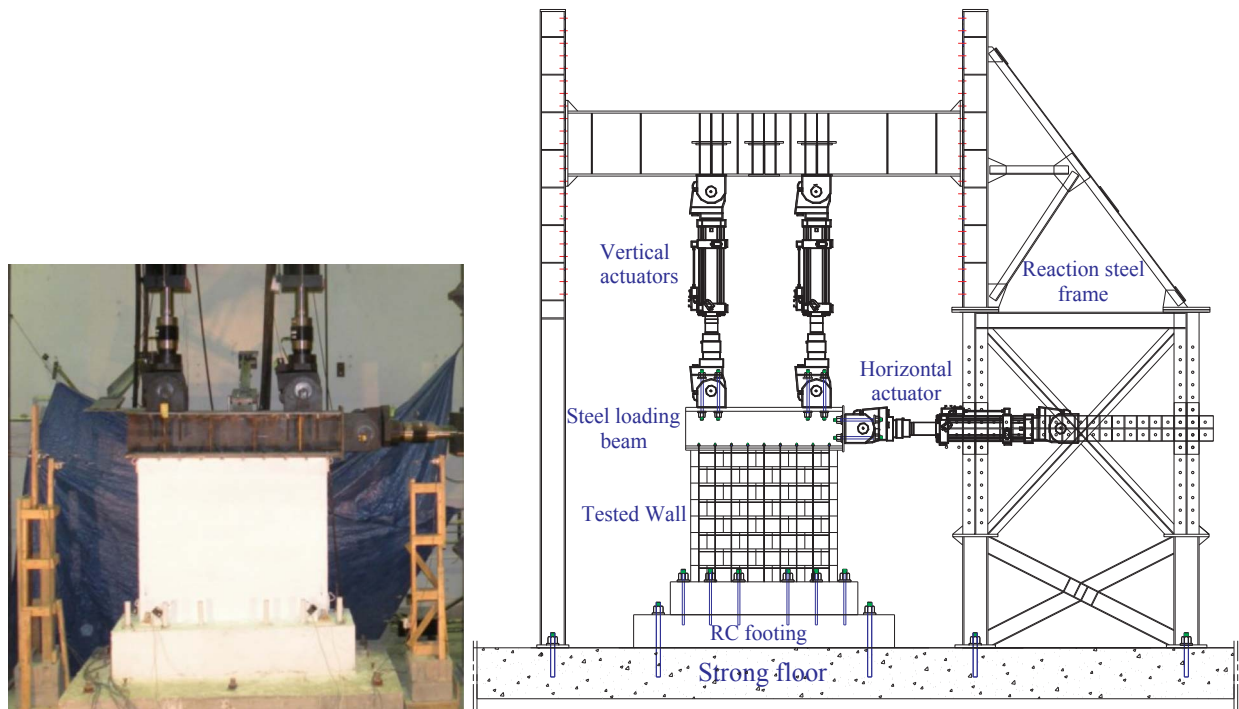


Fig. 4. Test setup for RM shear wall.

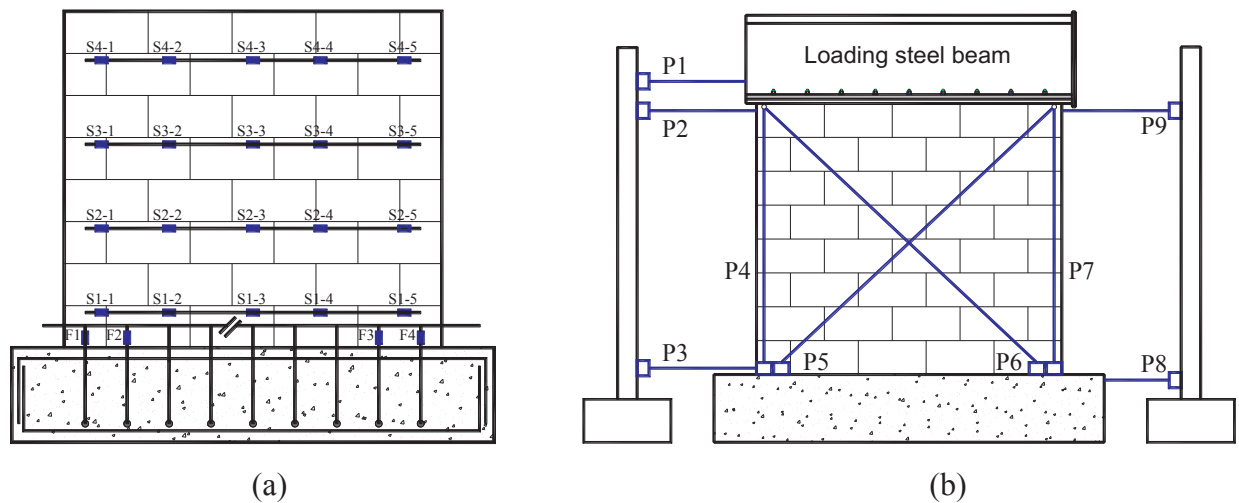


Fig. 5. Locations of: (a) strain gauges; (b) displacement potentiometers.

Fig. 6 presents the testing loading protocol. All the walls were tested under a constant axial compressive stress of 1.0 MPa that was applied using load-control protocol. Next, the test protocol was switched to displacement-control to introduce in-plane lateral displacements at the mid-height of the loading steel beam, according to the loading histories proposed by [15]. Two displacement cycles were completed for each target displacement increment. 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 as described in [16]. More details about the experimental work including the specimens and test methods used to experimentally measure the compressive strength of the block, mortar, and grout and the stress-strain relationship of the steel reinforcing bars, as well as the test-setup, the instrumentation and testing protocol are given by [17].

3. Test results

The hysteretic force-displacement response for each wall against the top drift is given in Fig. 7. Moreover, Table 2 summarizes the test results in both push and pull directions including the crack, yield, and ultimate capacities in addition to the drift limit of 1%. In this table, the lateral yield displacement, Δ_{ye} , was taken as the average between the top lateral displacements that are corresponding to the first yield in the vertical reinforcement in each direction; the lateral yield force, Q_{ye} , was defined at the lateral yield displacement, Δ_{ye} , on the backbone curve; Q_{ue} is the lateral peak load; Δ_{Que} is the lateral displacement at the peak lateral force, Q_{ue} ; $\Delta_{0.8Que}$ is the top lateral displacement defined at a drop in wall capacity to 80% of Q_{ue} ; and $\mu_{\Delta Que}$, $\mu_{\Delta 0.8Que}$, $\mu_{\Delta e1\%}$ are the lateral displacements ductility at Δ_{Que} , $\Delta_{0.8Que}$ and at a top drift of

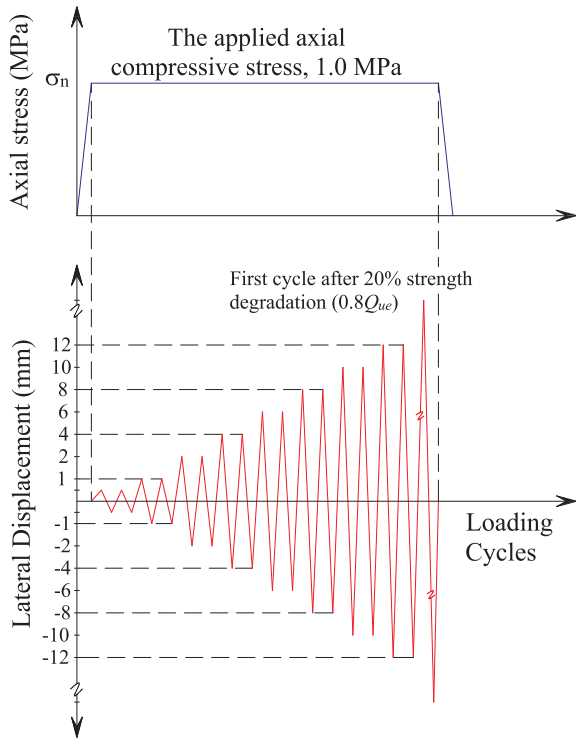


Fig. 6. Loading protocol.

1.0%, respectively. As shown in Fig. 7 and Table 2, the tested walls had a similar behavior in both push and pull directions with a general symmetric resistance. Thus, only the results in the push direction were considered for the evaluation of the studied parameters.

3.1. Effect of horizontal reinforcement anchorage end detail

Fig. 8 shows that the 180° hook is the most effective anchorage end detail in terms of lateral force capacity and ductility. However, wall W-90° reached a similar shear capacity as wall W-Ref with a 7.0% less displacement ductility, μ_Δ . Nevertheless, constructing wall W-Str with straight horizontal reinforcing bars resulted in a more reduction in both shear capacity and displacement ductility. Wall W-Str reached an ultimate lateral load, Q_{ue} , of 398 kN compared to 418 kN for wall W-Ref, followed by a rapid degradation in its in-plane lateral force resistance. At a 20% reduction in Q_{ue} , the straight horizontal reinforcement had the least ductile behavior with $\mu_{\Delta 0.8Q_{ue}}$ equal to 3.6; whereas, walls W-Ref and W-90° achieved $\mu_{\Delta 0.8Q_{ue}}$ of 4.2 and 3.9, respectively. The three tested walls W-Ref, W-90°, and W-Str were horizontally reinforced with four bars that were uniformly distributed along the height. The first bar was placed in the first course followed by 400 mm vertical spacing between bars. The readings of the lateral bars strain gauges show that the first bar from the base footing for wall W-Str did not have a sufficient development length to reach its yield strength. Therefore, the in-plane shear resistance provided by its horizontal reinforcement, V_s , just reached 84% of its yield capacity. On the contrary, providing 180° and 90° hook anchorage for walls W-Ref and W-90°, respectively, were more effective. Hence, all of their horizontal bars achieved their yield strength as shown in Fig. 9-a. This could explain the reduction in the

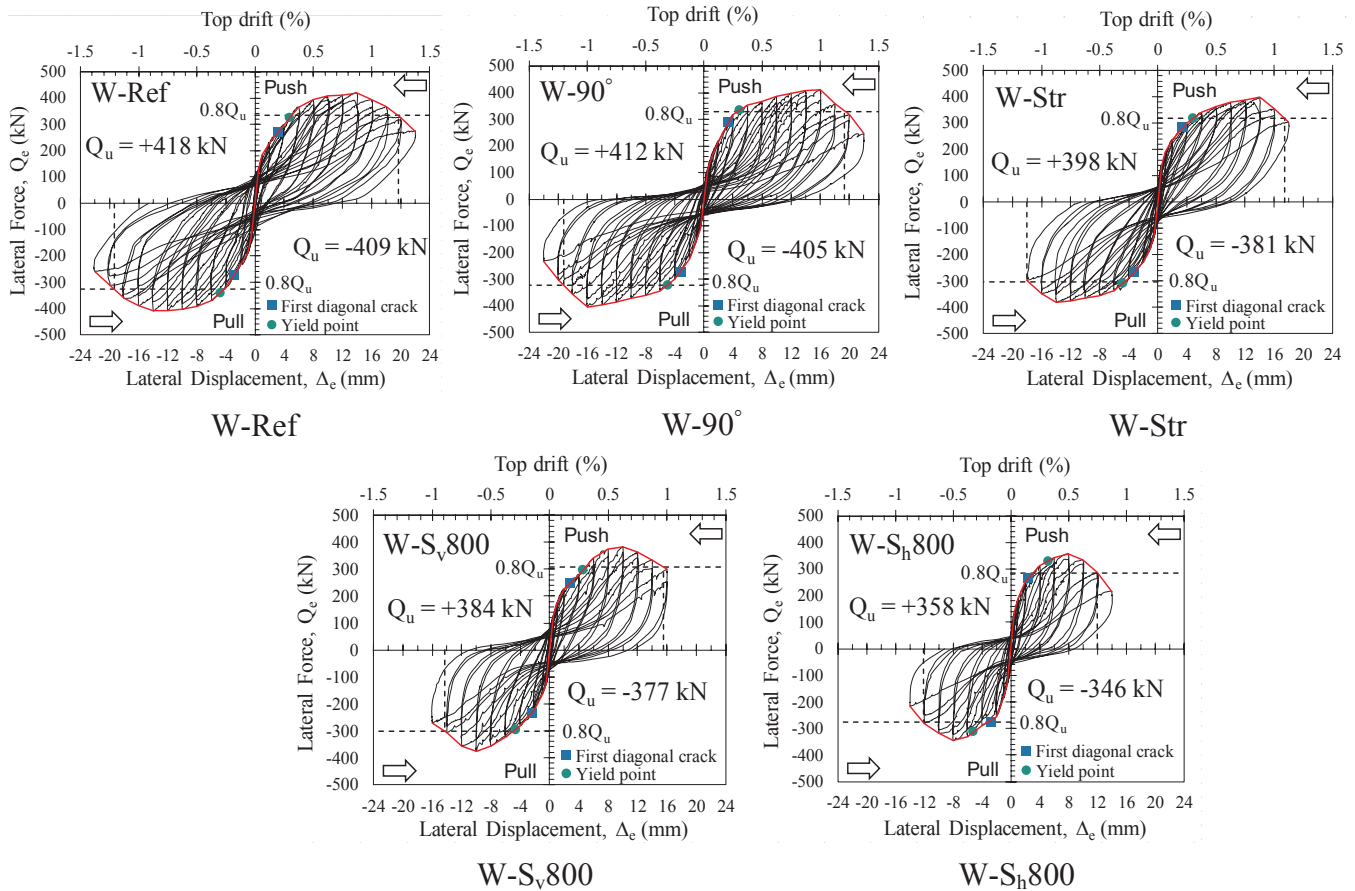
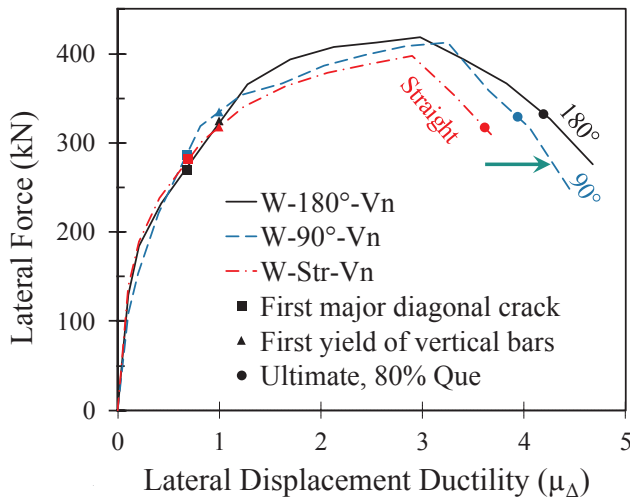


Fig. 7. Hysteretic force-displacement response of the five tested walls.

Table 2

Summary of experimental force and displacement capacities of tested walls.

Wall ID	Test results									
	Yield		Peak		Failure	Displacement ductility			First crack ^a	
	Q _{ye} (kN)	Δ _{ye} (mm)	Q _{ue} (kN)	Δ _{Que} (mm)	Δ _{0.8Que} (mm)	μ _{ΔQue} –	μ _{Δ0.8Que} –	μ _{Δe1%} –	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-90°	335	4.9	412	16.0	19.4	3.2	3.9	3.2	3.3	287
	–323	–4.9	–405	–16.0	–19.3	3.3	4.0	3.3	–3.1	–275
W-Str	318	4.8	398	14.0	17.4	2.9	3.6	3.3	3.4	281
	–310	–4.8	–381	–14.0	–18.0	2.9	3.8	3.4	–3.0	–268
W-S _v 800	300	4.6	384	10.0	15.6	2.2	3.4	3.5	2.8	248
	–294	–4.6	–377	–10.0	–14.3	2.2	3.1	3.5	–2.4	–234
W-S _h 800	330	5.2	358	8.0	12.0	1.5	2.3	N/A	2.6	265
	–313	–5.2	–346	–8.0	–12.1	1.5	2.3	N/A	–2.7	–275

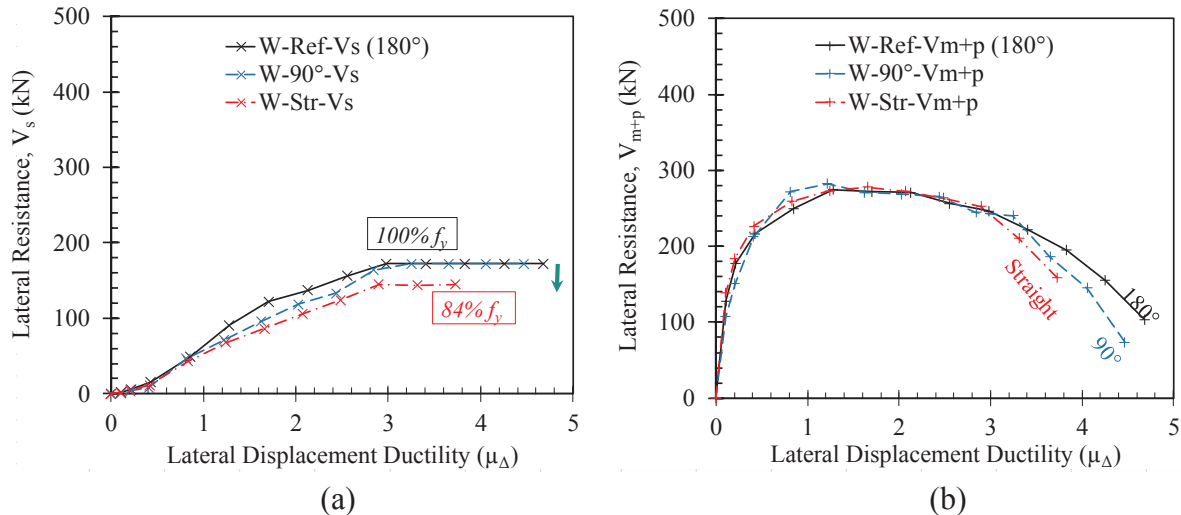
^a First major diagonal crack.**Fig. 8.** Effect of horizontal reinforcement anchorage end detail on the in-plane shear strength of RM shear walls.

reached ultimate lateral force, Q_{ue} , for wall W-Str.

Fig. 9-b presents the masonry and axial compressive stress contribution, V_{m+p} , at different levels of displacement ductility for the three tested walls. The slight variation in these relations show that the

horizontal reinforcement anchorage detail has a negligible effect on V_{m+p} up to the peak lateral force resistance, Q_{ue} ; however, it has a minor influence on the resistance degradation of V_{m+p} . Wall W-Ref had a gradual reduction in V_{m+p} compared to walls W-90° and W-Str. This slight enhancement could be attributed to the sufficient confinement of the 180° hook anchorage compared to the other end details. Fig. 10 shows that the 180° standard hook reduced the buckling length of the outer vertical bar compared to the 90° hook. Consequently, the outside core of the 180° hook end details has better confinement compared to the other end details.

It can be concluded from the test results of walls W-Ref, W-90°, and W-Str that the horizontal reinforcement anchorage end detail in RM shear walls has two main functions. The first one is to provide a sufficient development length such that the horizontal bars can reach their yield strength, while the second is to improve the confinement of the extreme vertical bars and the grout in the end zones under compressive stress. These two functions are responsible for the enhancement of the in-plane lateral strength and displacement ductility, respectively. Since the 90° and 180° hooks were able to provide the needed development length, the lateral resistance provided by the horizontal reinforcement in both walls reached its yield capacity. Even though both walls reached similar in-plane shear resistance, the 90° hook was not as efficient for confinement as the 180° hook. Consequently, wall W-Ref achieved higher displacement ductility. These conclusions could explain the seismic reinforcement requirements provided in CSA S304-14 [11],

**Fig. 9.** Effect of horizontal reinforcement anchorage end detail on shear resistance shares provided by: (a) horizontal reinforcement, V_s ; (b) masonry and axial compressive stress, V_{m+p} .

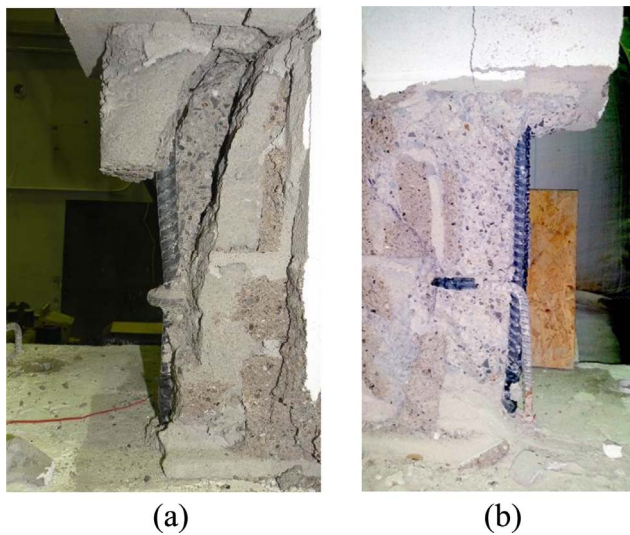


Fig. 10. In-plane lateral confinement of vertical reinforcement using: (a) 180° hook; (b) 90° hook anchorage end detail.

where a 180° hook is necessary inside the plastic hinge region of ductile shear walls due to the expected high level of plastic deformations. However, anywhere outside the plastic hinge region or inside the plastic hinge region for low ductile (moderately ductile) RM shear walls, a 90° hook is acceptable, where low level of deformations are expected. Although the 180° and 90° hooks are more efficient than the straight bar, using them may cause congestion at the end zone for narrow blocks. Moreover, the reduction in Q_{ue} when using straight horizontal bars (W-Str) is still minor, approximately 5%, when compared to 180° hooked bars (W-Ref). Therefore, to facilitate the construction process, it is suggested that straight horizontal bars could be permitted in moderately ductile squat shear walls.

The effect of horizontal reinforcement anchorage end detail on the crack pattern of the three tested walls at first major diagonal cracks, lateral peak load Q_{ue} , and when the lateral load dropped to 80% of Q_{ue} , is depicted in Fig. 11. In general, the walls constructed with 180° and 90° hooks had similar crack propagation, which is different from that of wall W-Str. The initial diagonal crack damage appeared during the +4.0 mm loading cycle for the three walls with a slight difference in the top drift. Walls W-Ref and W-Str reached their ultimate resistance at Δ_e equal to +14.0 mm followed by a degradation in the lateral resistance. However, Wall W-90° was able to achieve another

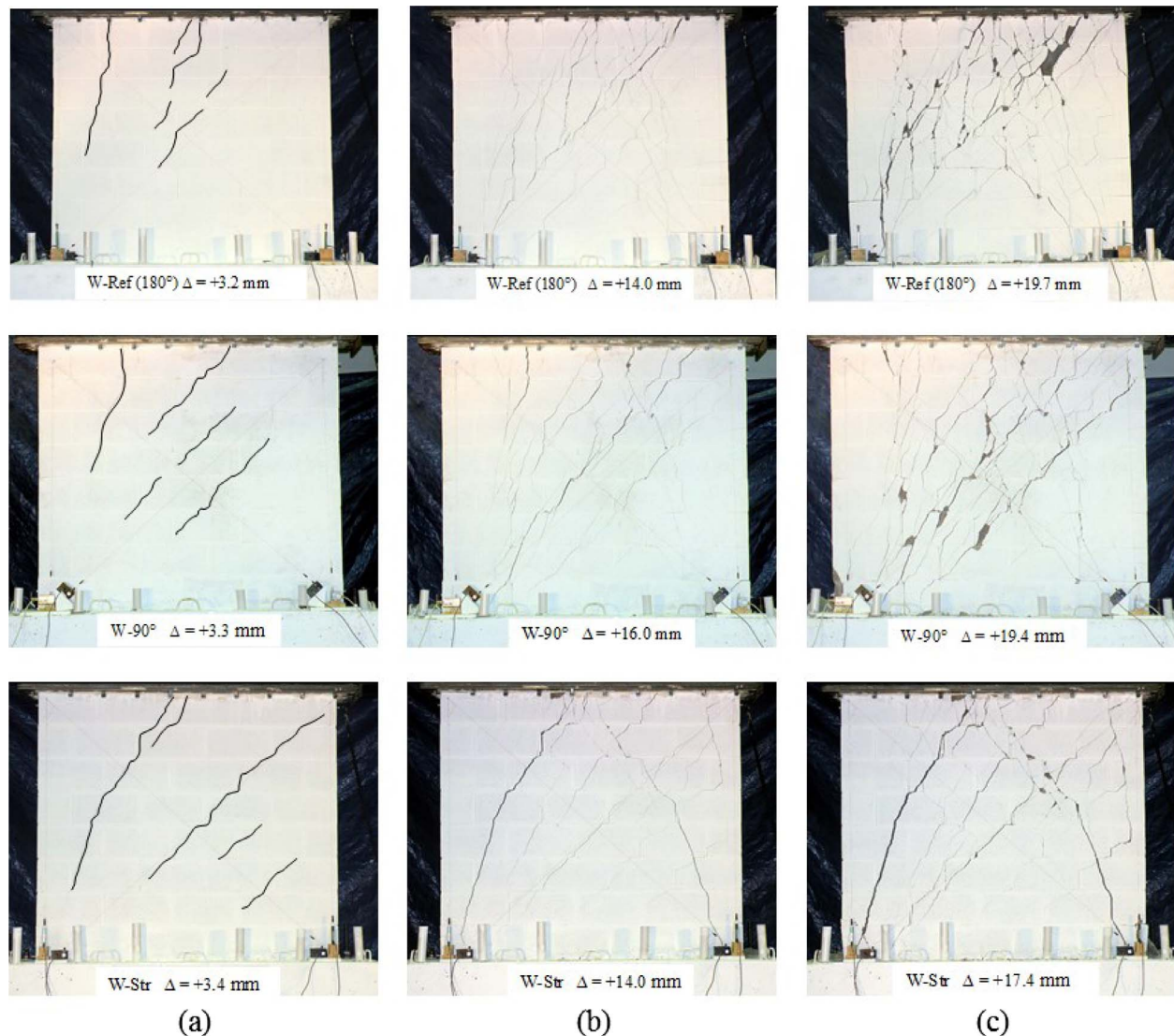


Fig. 11. Effect of horizontal reinforcement anchorage end detail on the crack pattern at: (a) first major diagonal cracks; (b) lateral peak load Q_{ue} ; (c) when the lateral load dropped to 80% of Q_{ue} .

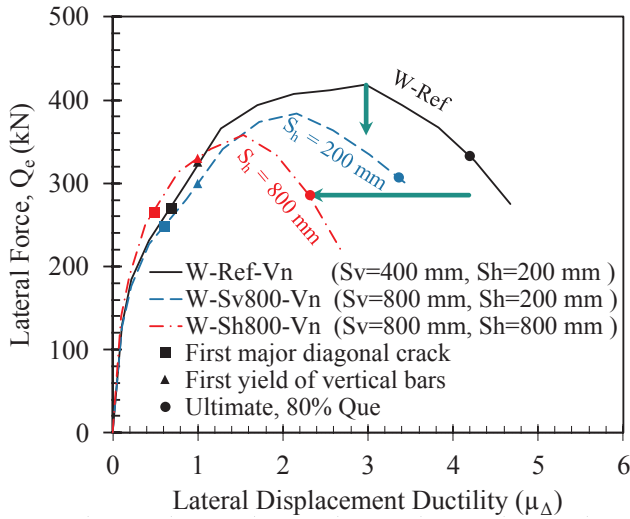
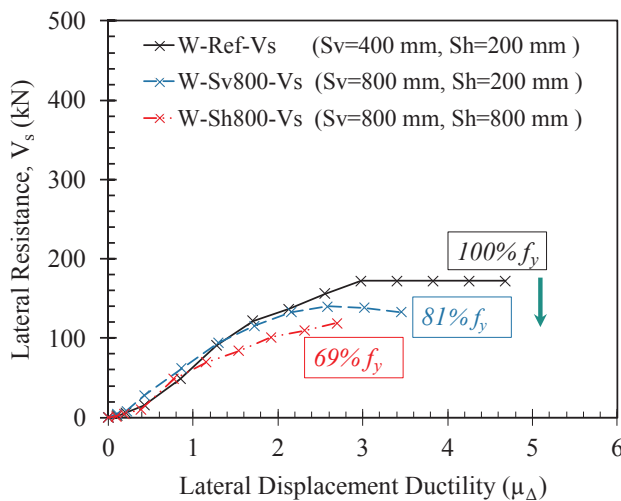


Fig. 12. Effect of the spacing of reinforcement on the in-plane shear strength of RM shear walls.

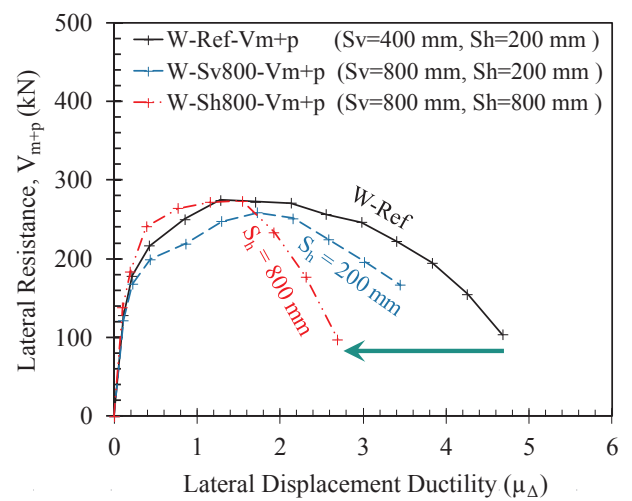
displacement increment before reaching its in-plane shear capacity with almost a constant resistance between Δ_e of +14.0 and +16.0 mm. At the peak load, Q_{ue} , wall W-Str had two major diagonal cracks that started from the middle length of the top to the bottom corners. These main cracks were accompanied by a small number of minor cracks. However, anchoring the lateral reinforcement at the end of walls W-Ref and W-90° resulted in sets of minor cracks along the diagonal axes, instead of the major diagonal cracks observed in wall W-Str. Increasing the imposed top lateral displacement resulted in the major diagonal cracks of wall W-Str became wider and they went through the thickness of the wall. The cracks in walls W-Ref and W-90° did not widen significantly; instead, face shell spalling was observed at drift limit of 1.1%. As a conclusion, not providing a sufficient development length for the horizontal bars at the ends of the tested wall resulted in a sort of a brittle failure.

3.2. Effect of the spacing between the horizontal and vertical reinforcement (S_v and S_h)

Although Walls W-Ref, W-Sv800, and W-Sh800 have the same



(a)



(b)

Fig. 13. Effect of spacing of reinforcement on shear resistance shares provided by: (a) horizontal reinforcement, V_s ; (b) masonry and axial compressive stress, V_{m+p} .

theoretical in-plane shear strength, V_n , according to most of masonry design codes, such as CSA S304-14 [11] (Table 1), the experimental results show a significant reduction in Q_{ue} and $\mu_{\Delta e}$ by increasing the spacing of the vertical and horizontal bars. Fig. 12 presents the effect of the spacing of reinforcement on the in-plane shear strength of the tested walls. Using horizontal reinforcement of 15 M@800 mm instead of 10 M@400 mm, with the same ρ_h of 0.13%, resulted in a reduction in Q_{ue} from 418 to 384 kN. Moreover, wall W-Sv800 reached a displacement ductility, $\mu_{\Delta 0.8Q_{ue}}$, of 3.4 compared to 4.2 for wall W-Ref. Yet, concentrating the vertical reinforcement in the first, middle, and last cells, as in wall W-Sh800, instead of distributing it in each cell with smaller areas, as in wall W-Sv800, led to a more brittle behavior accompanied with less strength. Wall W-Sh800 had Q_{ue} and $\mu_{\Delta 0.8Q_{ue}}$ equal to 358 kN and 2.3, respectively. These values are approximately 93% and 68% of the corresponding measured values for wall W-Sv800 and 85% and 55% for wall W-Ref, respectively. Wall W-Sv800 reached in-plane lateral resistance of 265 kN at a displacement of $0.62\Delta_y$ before the major diagonal cracks occurred compared to 248 kN and $0.6\Delta_y$ for wall W-Sh800 and 272 kN and $0.68\Delta_y$ for wall W-Ref, respectively. These results show that the distribution of the reinforcement does not have a significant influence on the pre-crack behavior compared to its considerable effect on the post-crack performance.

For a better understanding of the behavior of the three tested walls, the in-plane shear resistance contributions provided by horizontal reinforcement, V_s , and masonry and axial compressive stress, V_{m+p} , were evaluated separately and are given in Fig. 13. Increasing the spacing of reinforcement resulted in a reduction in the horizontal reinforcement contribution, which was responsible for the losses in Q_{ue} . Constructing Wall W-Ref with uniformly distributed vertical and horizontal reinforcement with small spacing allowed the lateral reinforcement to achieve its yield capacity. However, $V_{s,max}$ only reached $0.81f_y$ when the spacing between the bond beams in wall W-Sv800 was increased to 800 mm. Moreover, the results of wall W-Sh800 show that the spacing of vertical reinforcement has an indirect impact on the contribution of V_s . Although both walls W-Sv800 and W-Sh800 had the same distribution of the horizontal reinforcement, 15 M@800 mm, V_s was not able to contribute more than 69% of its yield resistance in wall W-Sh800 compared to 81% of f_y in wall W-Sv800 as shown in Fig. 13-a. This reduction in V_s is due to the redistribution of the vertical reinforcement since the 20 M@200 mm vertical reinforcement in wall W-Ref was replaced by 30 M vertical bars with a spacing of 800 mm in wall W-Sh800.

By subtracting the provided horizontal reinforcement resistance, V_s ,

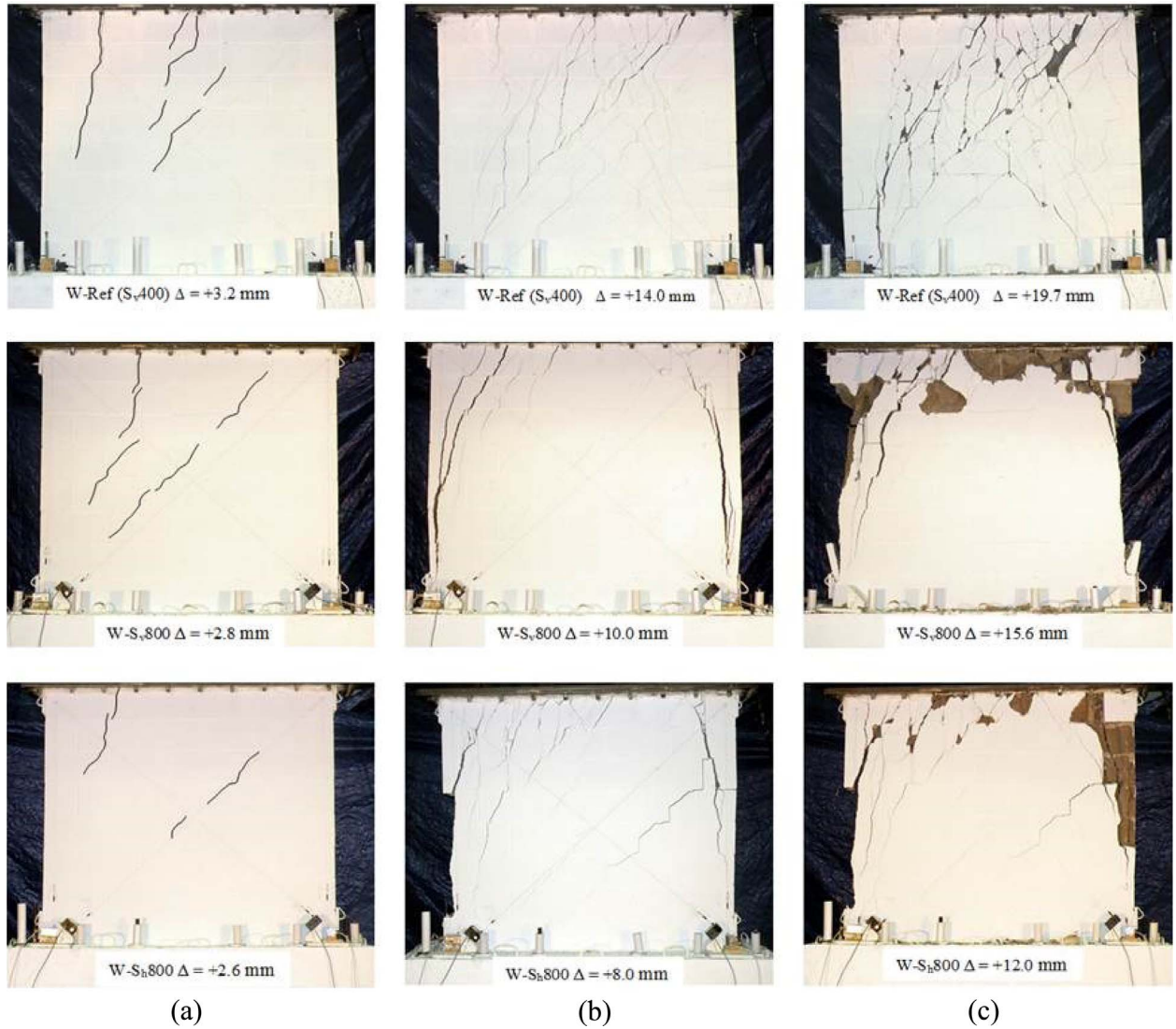


Fig. 14. Effect of the spacing of reinforcement on the crack pattern at: (a) first major diagonal cracks; (b) lateral peak load Q_{ue} ; (c) when the lateral load dropped to 80% of Q_{ue} .

from the measured lateral force, the combined contribution of the masonry and axial compressive stress, $V_{(m+p)}$, was calculated for each wall and is presented in Fig. 13-b. As shown in this figure, the three walls almost reached the same $V_{(m+p)max}$ at close displacement ductility values. The overall loss in the displacement ductility, $\mu_{\Delta 0.8Q_{ue}}$, from 4.2 to 3.4 and 2.3 for walls W-Ref, W-S_v800, and W-S_h800, respectively, could be attributed to the rapid degradation in $V_{(m+p)}$ in walls W-S_h800 and W-S_v800 compared to the more gradually reduction in wall W-Ref.

Masonry walls have directional properties that vary according to the orientation of the mortar joints relative to the applied principal stresses. The presence of grout and reinforcement enhance this behavior by creating continuity in the stress flow. Although all tested walls were constructed using knock-out concrete masonry units to provide some sort of grout continuity in the horizontal direction, the distribution of horizontal and vertical reinforcement still had a major impact on stress flow. This can be seen from the shear crack penetration as depicted in Fig. 14. Concentrating the horizontal reinforcement at the top and bottom of wall W-S_v800, or concentrating the lateral and vertical reinforcement in wall W-S_h800, leads to a localized failure mechanism at the boundaries of the tested walls. Unlike wall W-Ref that reached a top lateral displacement, Δ_e , of 19.7 mm without a significant crushing or spalling of the face shell, wall W-S_v800 had completely separated parts at an earlier imposed Δ_e equals to +15.6. Moreover, wall W-S_h800 lost more than 40% of its in-plane shear capacity at target displacement

$\Delta_e = +14.0$ mm with the instability of the loading system, which led to the termination of the test. However, the final crack pattern of wall W-S_v800 in Fig. 15 shows the discontinuity in the stress flow since no significant damage occurred in the main core of grout compared to the severe damage to the wall ends.

Stiffness degradation, energy dissipation, and equivalent viscous damping are important aspects that need to be considered when evaluating the seismic performance of RM shear walls, as well as when modeling their cyclic response. The secant stiffness at any loading cycle, $K_{s,i}$, was defined as the ratio between the lateral resistance, Q_i , and the corresponding top lateral displacement, Δ_i . The initial gross stiffness, K_g , was calculated at the first cycle of ± 0.5 mm. Hence the secant stiffness degradation can be calculated as K_{sc}/K_g . The measured secant stiffness, K_{sc} , and the stiffness degradation, K_{sc}/K_g , of walls W-Ref, W-S_v800, and W-S_h800 at different levels of deformations are given in Fig. 16.

The three walls had almost the same initial gross stiffness, K_g , with similar force-displacement response up to a top drift of 0.25%; thereafter, wall W-S_h800 had a significant degradation in its secant stiffness compared to wall W-Ref. At a drift limit of 1.0%, wall W-Ref had a secant stiffness of $0.095K_g$. Increasing the spacing between the horizontal reinforcement from 200 to 800 mm resulted in more reduction in the wall's secant stiffness. At the same top drift, K_{sc} of wall W-S_v800 was measured as $0.074K_g$, which means a 25% loss in the corresponding

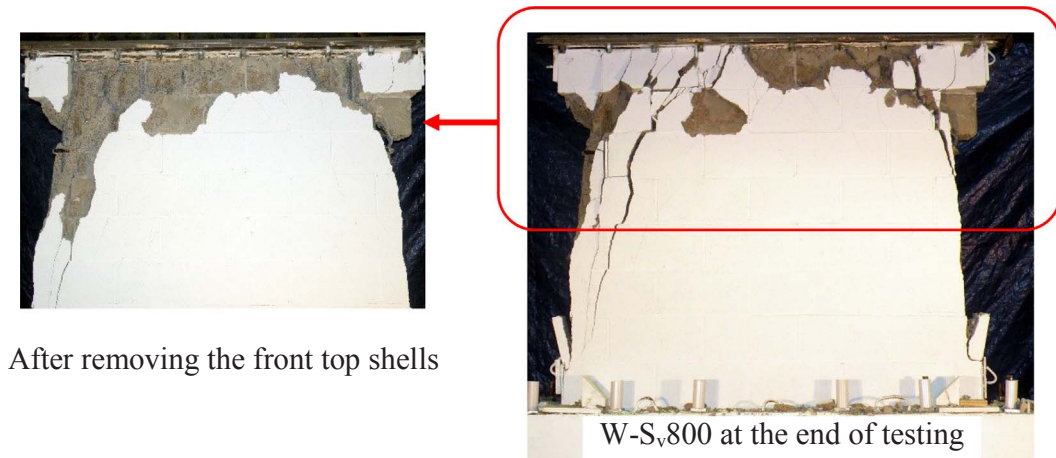
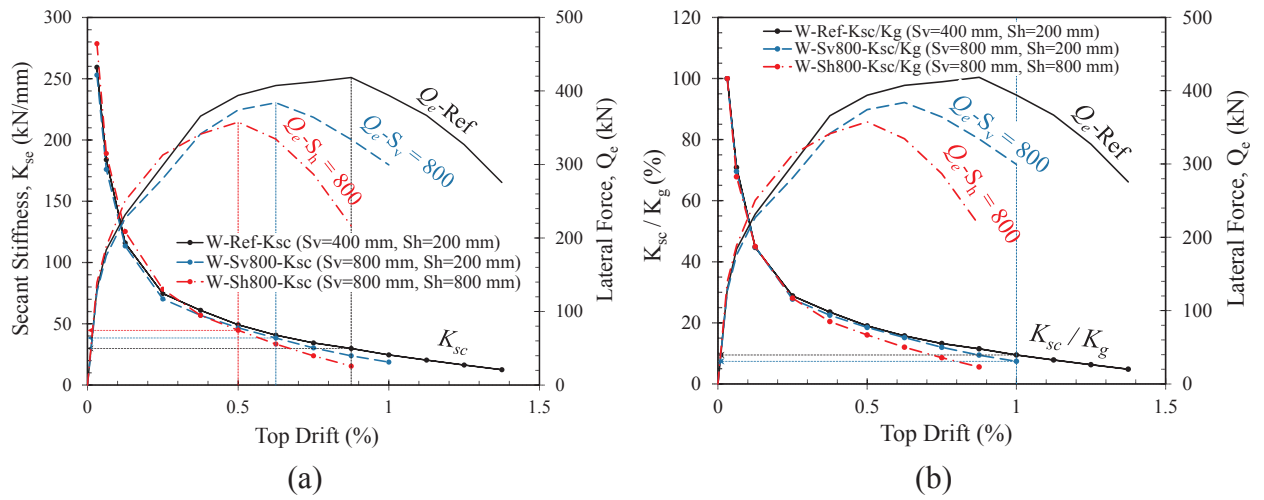


Fig. 15. Final crack pattern of tested wall W-Sv800.

Fig. 16. Effect of the spacing of reinforcement on stiffness degradation: (a) K_{sc} ; (b) K_{sc}/K_g .

lateral force resistance, Q_e . When the spacing of vertical reinforcement was also increased, wall W-Sh800 was not even able to achieve a drift limit of 1.0%. However, it had stiffness degradation, K_{sc}/K_g equal to 0.1, at a top drift of 0.7% compared to wall W-Ref that reached the same degradation at top drift equal to 1%.

The capacity of shear walls to dissipate energy is another important aspect of seismic design and in the analysis of their cyclic response. The energy dissipation, E_d , was defined as the area enclosed within the inelastic hysteretic force-displacement response, as proposed by [18]. However, the elastic stored strain energy, E_s , was calculated as the area under the equivalent linear elastic response. The energy dissipation within different structure systems at the inelastic behavior can be quantified through hysteric damping. The hysteric damping described by [19] as an equivalent viscous damping ratio, ζ_{eq} , using an equal area approach by equating the energy dissipated by a viscous damper with the energy dissipated from non-linear behavior using the following equation:

$$\zeta_{eq} = \frac{1}{4\pi} \frac{E_d}{E_s} \quad (1)$$

Damping is generally specified for the whole structure rather than for an individual element. However, most RM structures are typically constructed with RM shear walls that are connected together by rigid diaphragms. Consequently, the trend of damping for a structural element such as shear walls, with respect to the top drift or the displacement ductility, can provide an indication of the overall response of RM

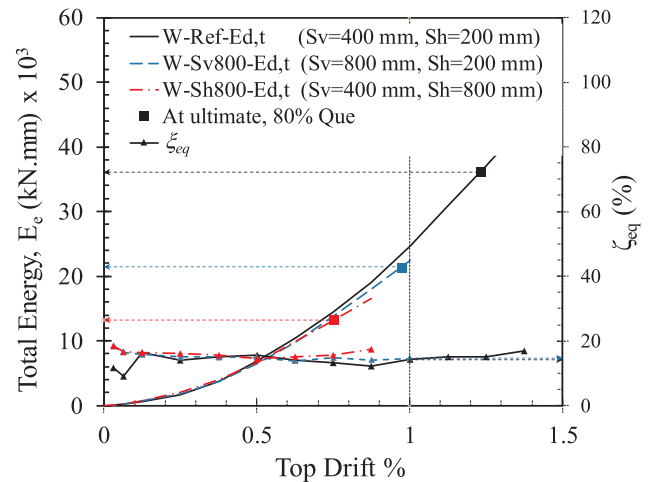


Fig. 17. Effect of the spacing of reinforcement on energy dissipation.

structures. The spacing of reinforcement shows a significant impact on the total energy dissipated by tested RM shear walls under in-plane cyclic lateral excitations as shown in Fig. 17. At 0.75% top drift, the three walls had the same total dissipated energy. Nevertheless, wall W-Sh800 was suffering extensive damage while wall W-Ref had a set of minor cracks. Distributing the vertical reinforcement in each cell

relatively enhanced the behavior of wall W-S_h800 compared to wall W-S_h800 and resulted in higher energy dissipation. However, wall W-Ref was able to dissipate approximately 2.7 and 1.7 times the energy dissipated by walls W-S_h800 and W-S_h800, respectively, when reaching the same drop in the lateral capacity, $0.8Q_{ue}$. This enhancement in the efficiency of dissipating energy was a direct result of distributing the horizontal and vertical reinforcement with small spacing.

3.3. Requirements for seismic reinforcement according to Canadian standard CSA S304-14 [11]

The Canadian Standards Association CSA S304 for the design of masonry structures classifies RM shear walls according to their ductility. The previous version CSA S304.1-04 [20] classified it as follows: conventional construction shear walls ($R_d = 1.5$); limited ductility shear walls ($R_d = 1.5$); and moderately ductile shear walls ($R_d = 2.0$). However, modifications have been made to this classification in the current version, CSA S304-14, such that the limited ductility shear wall was removed and a new class introduced for ductile shear walls ($R_d = 3.0$). The plastic hinge region was defined for moderately ductile and ductile RM shear walls, with R_d equal to 2.0 and 3.0, respectively. In this region, additional seismic reinforcement detailing is required where inelastic deformations occur. According to the seismic reinforcement requirements for moderately ductile RM shear walls in CSA S304-14 [11], the anchorage of the horizontal reinforcing bars in the plastic hinge region shall have a 90° or more standard hook at the ends of the conventional and moderately ductile walls, whereas a 180° standard hook is required for the ductile shear walls. However, in the previous CSA S304.1-04 [20], only a 180° standard hook was mandatory for ductile wall classes, including the limited ductile. For the conventional construction RM shear walls with R_d equal to 1.5 and outside the plastic hinge region for others R_d values, a 90° standard hook is required. However, some international codes (e.g. New Zealand, NZS 4230:2004 [21]) accept a 90° anchorage hook for ductile RM shear walls.

The current version CSA S304-14 [11], defines the bond beam as a course or courses of a masonry wall grouted and reinforced in the horizontal direction with reinforcing bars. The bond beam may serve as a horizontal wall tie, as a bearing surface for structural element member, or as a beam. The joint reinforcement commonly refers to the embedded horizontal reinforcement in the mortar joints. The shear reinforcement resistance, V_s , in RM shear walls is mainly provided by the horizontal steel bars in the bond beam and/or the joint reinforcement. However, the joint reinforcement is more common for partially grouted RM shear wall, which is beyond the scope of this paper. In general, the Canadian standard CSA S304-14 [11] limits the maximum spacing between the horizontal reinforcement, S_v , for seismic design to 1200 mm where only bond beams are used; 400 mm where only joint reinforcement is provided; or 2400 mm for bond beams and 400 mm for joint reinforcement where both are used. However, S_v shall not exceed $0.5l_w$ for bond beams. In addition, the maximum spacing of vertical reinforcement S_h , shall be the lesser of $12(t + 10)$ mm or 2400 mm for seismic hazard index, $I_E F_a S_a(0.2)$, less than 0.75; or half of these values if $I_E F_a S_a(0.2) \geq 0.75$, where I_E , F_a , $S_a(0.2)$ are defined in the National Building Code of Canada (NBCC 2010) [22]. For better seismic performance, S_v is limited to 600 mm instead of 1200 mm inside the plastic hinge region of ductile shear walls ($R_d = 3.0$). Moreover, for moderately ductile and ductile RM shear walls, $S_{h,max}$ shall be taken as $6(t + 10)$ mm, 1200 mm, or $0.25l_w$, whichever is less, whatever the value of the seismic hazard index. However, $S_{h,max}$ need not be less than 600 or 400 mm where the spacing required for strength is greater than 600 or 400 mm for R_d equals 2.0 or 3.0, respectively. These minimum limits are provided to ensure flexural failure mode.

Unlike the seismic reinforcement requirements provided in CSA S304-14 [11], the Standards Association of New Zealand NZS 4230:2004 for the design of reinforced concrete masonry structures

provide more conservative limits for the spacing of reinforcement. According to NZS 4230:2004 [21], the spacing between the vertical reinforcement inside the plastic hinge region should not be taken more than 400 mm with a minimum of 4 vertical bars. In addition, it should be uniformly distributed along the length of the wall. Moreover, the horizontal reinforcement shall be uniformly distributed throughout the height of the wall with a maximum spacing of 400 mm in the case of a RM building with not greater than 3 stories or 12 m height. This maximum limit becomes 200 mm in walls greater than 3 stories or 12 m in height. Finally, from the results of the tested walls it can be concluded that the distribution of reinforcement has a significant effect on the in-plane shear capacity, with a major impact on the achieved displacement ductility. Hence, it is recommended that the current values for the maximum spacing of reinforcement in the Canadian standard CSA S304-14 [11] be modified by specifying lower spacing limits.

4. Summary and conclusions

This paper assesses experimentally the in-plane shear behavior of five single-story RM shear walls when subjected to in-plane axial compressive stress and cyclic lateral excitations. The studied parameters were the horizontal reinforcement anchorage end details and the spacing of the horizontal and vertical reinforcement. From the test results, the 180° standard hook was the most efficient in terms of strength and ductility compared to the 90° hook that had similar strength with less ductility. On the contrary, the tested wall with straight horizontal bars reached the smallest values for strength and displacement ductility among the three anchorage details. However, the reduction in Q_{ue} when using straight horizontal bars (W-Str) is still minor, approximately 5%, when compared to 180° hooked bars (W-Ref). Therefore, to facilitate the construction process, it is suggested that straight horizontal bars could be permitted in moderately ductile squat shear walls. The spacing of the reinforcement has a considerable effect on the stress flow, and thereby on the behavior of RM shear walls. Tested walls that were constructed with closely spaced reinforcement were able to reach higher strength and ductility and dissipate higher energy compared to similar walls with large spacing when using the same reinforcement ratio, which agrees with the results of previous research. Hence, the current values for the maximum spacing of reinforcement in the Canadian standard CSA S304-14 [11] need to be modified by specifying lower spacing limits.

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