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REPAIR OF EARTHQUAKE DAMAGED SQUAT REINFORCED CONCRETE SHEAR WALLS USING EXTERNALLY BONDED CFRP SHEETS

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Abstract

This study investigates the effectiveness of using externally bonded carbon fiber-reinforced polymer (CFRP) sheets to prevent premature shear failure and improve the seismic performance of deficient squat reinforced concrete (RC) shear walls. The wall specimens are designed using older less-stringent design standards [CSA A23.3-77; ACI 318-68] to replicate low-rise shear walls representative of construction practices during the 1960s and 1970s. The deficient design details in the specimens include insufficient shear reinforcement, a lack of concrete confinement at the ends of the shear wall and lap splices of the longitudinal reinforcement in the potential plastic hinge region. The shear wall specimens described in this study are tested first under reversed cyclic load to simulate the damage and drift effects of an earthquake on the wall. Externally bonded CFRP sheets are applied to the surfaces of the specimens as a minimally disruptive rehabilitation strategy for walls that have been damaged during an earthquake and need to be repaired. The performance of the retrofitting strategy is evaluated in terms of its potential to improve the lateral-load carrying capacity, ductility and energy dissipation capacity of the specimens. Experimental results show that squat RC shear walls with detailing deficiencies are susceptible to brittle modes of failure, including diagonal tension shear failure and lap splice splitting failure. In a deficient wall specimen without lap splices, the CFRP rehabilitation is shown to be successful in avoiding brittle shear related failures and allowing the flexural steel reinforcement to yield resulting in improvements in lateral load carrying capacity. In a specimen with sufficient lap splice length, the CFRP rehabilitation strategy is shown to be effective in completely restoring the in-plane strength of the specimen while also slightly improving the ductility and energy dissipation capacity.

Keywords: reinforced concrete; shear wall; lap splice; cyclic loading; seismic retrofit; carbon fiber-reinforced polymer

1. Introduction

Squat reinforced concrete (RC) shear walls, with a height-to-length aspect ratio (h_w/l_w) of less than 2, are a common lateral load resisting structural system used in low-rise buildings such as industrial and nuclear facilities around the world. The seismic design methodologies for shear wall systems are based on a life-safety approach. That is, ensuring that the shear walls have the appropriate levels of stiffness, strength, and ductility to withstand a major seismic event without reaching the collapse state, thus preventing loss of life. Modern design standards for earthquake resistant design of RC structures allow engineers to design RC shear walls with the appropriate levels of stiffness and strength. With proper detailing of longitudinal and transverse steel reinforcement, shear walls can exhibit a dependable ductile response, with sufficient energy dissipation capacity to survive a major earthquake [1]. However, there is still a large stock of existing low-rise shear wall structures around the world that have been designed with old design standards containing detailing deficiencies. The common detailing deficiencies include insufficient shear reinforcement, no additional concrete confinement at the two ends of the wall, and lap splices of the longitudinal steel reinforcement located in the potential plastic hinge region. In addition, shear walls with a height-to-length aspect ratio (h_w/l_w) of less than 2 typically exhibit a combination of flexure and shear behavior. The combination of the shear dominant nature of squat RC walls and the detailing deficiencies in the design of older walls can lead to undesirable brittle modes of failure associated with shear. To retrofit these deficient structures, a detailed understanding of their behavioral mechanisms is crucial.

In general, there are three commonly recognized failure modes associated with shear: (1) diagonal tension shear failure, (2) diagonal compression shear failure, and (3) sliding shear failure [1]. Figure 1 illustrates each failure mode described herein. Low aspect ratio shear walls experience high shear stresses under lateral loads and when adequate shear reinforcement is not provided, these types of walls are susceptible to diagonal tension shear failure. Under in-plane loading cracks form in the concrete in the principle tensile stress direction, which is along the diagonal of a shear wall element. At the location of these cracks the horizontal steel reinforcement is responsible for transferring shear stresses across the cracks in the concrete. When an insufficient amount of horizontal steel reinforcement is provided, the application of additional lateral load will cause the horizontal steel reinforcement to rupture and lead to failure of the wall in what is referred to as diagonal tension shear failure. As shown in Fig. 1a, diagonal tension shear failure is often characterized by a distinct diagonal failure plane forming from one corner of the wall to the other. Diagonal tension shear failure is considered an undesirable mode of failure because it occurs in a sudden and brittle manner, giving very little warning of impending failure to the occupants of a structure. Alternatively, when adequate steel shear reinforcement is provided, diagonal compression struts in the concrete (Fig. 1b) are responsible for transferring large compressive forces to the adjacent structural element. As illustrated in Fig. 1b, high compressive forces in the compression struts can lead to crushing of the concrete at the toe of the wall. Degradation of the concrete at the toe of the wall leads to losses in strength and stiffness under subsequent load reversals. If adequate concrete confinement is not provided at the two ends of the shear wall, crushing at the toe of the wall can lead to buckling of the flexural steel reinforcement and out-of-plane instability leading to a loss in load carrying capacity.

When adequate shear reinforcement is provided and the stress in the compression struts is limited to less than the crushing stress of the concrete, the tendency for a shear wall to experience diagonal tension or

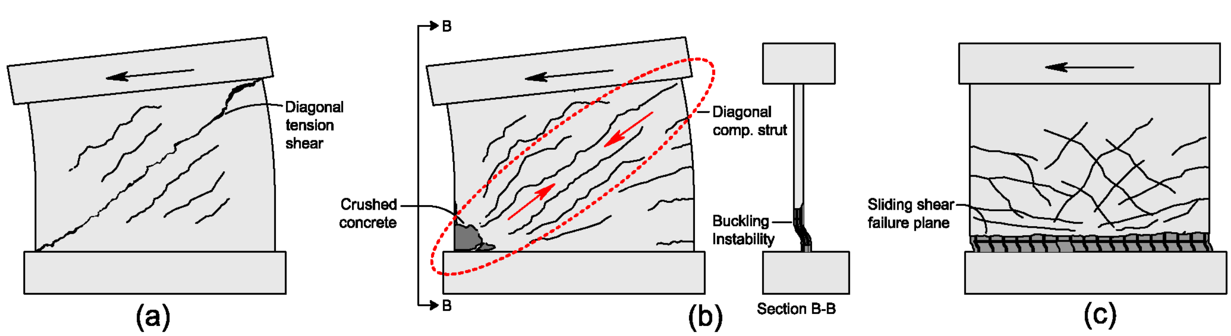


Fig. 1 – Shear wall failure modes: (a) diagonal tension; (b) diagonal compression; (c) sliding shear

compression shear failure is controlled. In such cases, it is expected that the flexural reinforcement will yield along the base of the wall forming a horizontal crack at the interface between the wall and the adjacent structural element. Once this crack extends along the entire length of the shear wall, dowel action of the flexural reinforcement and aggregate interlock between the concrete surfaces are responsible for transferring the shear stress to the adjacent structural element. Under cyclic loading, degradation of the aggregate interlock between the two surfaces, as illustrated in Fig. 1c, can lead to kinking of the flexural steel reinforcement and large translational motion of the wall causing a loss in load carrying capacity.

When flexural steel reinforcement is lapped or spliced along the base of a shear wall, this introduces another potential mode of failure in addition to flexural failure or those associated with shear. Prior to the introduction of modern seismic design provisions, older design guidelines permitted the use of lap splices in critical locations where significant inelastic deformations are anticipated, known as the plastic hinge region. Even when sufficient lap splice length is provided, lap splices located in the plastic hinge region have been shown to slip [1]. Slipping between steel reinforcing bars in a lap splice can occur in two potential modes: (1) pullout failure of the concrete surrounding the lap splice, and (2) splitting failure, caused by cracks in the concrete along the length of the lap splice. The tendency for a lap splice to experience a particular mode of failure depends on the amount of concrete confinement around the lap splice. When adequate confinement is provided in the form of transverse hoops or rectangular ties, pullout failure is more likely to occur because confining pressure restrains any relative motion between the lapped bars. As illustrated in Fig. 2a, as the lapped bars are loaded in tension a failure plane forms around the outer diameter of the rebar ribs after the mechanical interlock between the rebar ribs and the concrete keys deteriorates. Alternatively, in poorly confined lap splices the bars are able to move relative to one another because of a lack of confining pressure resulting in a splitting crack oriented along the length of the lapped bars. As shown in Fig. 2b, under increasing load the splitting crack will dilate until the lapped bars slip relative to one another. Once splitting failure occurs, there is no longer any mechanical interlock between the rebar ribs resulting in an incompatibility in transferring the forces to the adjacent structural element and the wall specimen is not able to achieve its full moment capacity. In this scenario, the strength of the shear wall is governed by the residual capacity of the lap splice.

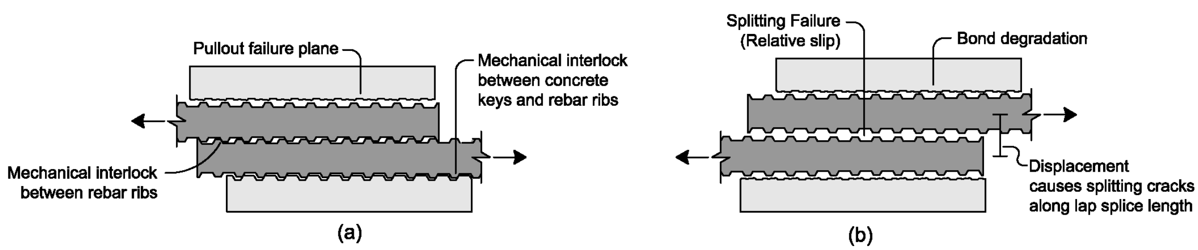


Fig. 2 – Lap splice failure mechanisms: (a) pullout failure; (b) splitting failure

Modern design guidelines for RC structures allow engineers to control the governing mode of failure, avoiding brittle failure modes associated with shear through proper detailing of longitudinal and horizontal steel reinforcement. However, many existing older RC shear wall structures have been designed with little to no consideration for earthquake induced lateral loads. As a result, there are a number of deficiencies in their design that have been shown during recent earthquake events to lead to brittle shear related failure, with very minimal ductility or energy dissipation capacity. To improve the seismic performance of older deficient shear wall structures this study investigates a retrofitting strategy consisting of externally bonded carbon fiber-reinforced polymer (CFRP) sheets. The use of CFRP sheets presents a practical and minimally disruptive retrofitting solution for RC structures and has been extensively studied in retrofitting applications for RC beams and columns. In RC shear walls, studies have been largely limited to the use of CFRP jackets to improve shear strength and confinement of the boundary elements in shear walls with aspect ratios between 1.0 and 1.5 [2, 3]. The use of CFRP sheets to improve the capacity of RC elements with lap splices has been largely limited to RC columns. In RC shear walls with lap splices, Patterson and Mitchell (2003) studied the use of CFRP wraps and headed bars to improve the seismic performance of deficient flexural walls ($h_w/l_w=2.7$ & 3.2) with adequate lap splice length [4]. The proposed retrofitting strategy is shown to be successful in improving the ductility and energy dissipation capacity in flexural walls. A study by Layssi et al. (2012) expanded on previous research by Paterson and Mitchell (2003) and studied the effectiveness of using CFRP wraps for flexural walls ($h_w/l_w=2.7$) with insufficient lap splice length in addition to poor confinement and shear deficiencies [5]. The study found that the use of CFRP jackets is successful in delaying failure of the lap splice and allows some yielding in the flexural reinforcement while preventing brittle shear failure.

The goal of this study is to expand on previous research, and understand the effectiveness of using a minimally disruptive retrofitting solution consisting of externally bonded CFRP sheets to improve the seismic performance of "squat" RC shear walls ($h_w/l_w=0.65$) with a without lap splices. To improve the practicality and reduce the intrusiveness of the retrofitting strategy, the CFRP sheets are not wrapped around the wall. Although the application of CFRP jackets around existing columns has been found to be effective, there are cases for RC shear walls when the ends of the wall are not exposed making it impossible to wrap the CFRP sheets around the wall. Past studied by Lombard et al. (2000), Hiotakis (2004), Cruz-Noguez et al. (2014) and Woods et al. (2016) have investigated the use of CFRP sheets applied only to the faces of the shear wall, thereby not improving the confinement at the ends [6, 7, 8]. The studies include a range of test specimens with different aspect ratios ($h_w/l_w=1.2$ & 0.85), including those designed with or without adequate seismic detailing. This study expands on previous research and includes low aspect ratio squat walls ($h_w/l_w=0.65$) with detailing deficiencies associated with older design standards [9, 10]. The deficiencies in their design include insufficient shear reinforcement, no concrete confinement at the ends of the wall, and lap splices of the longitudinal steel reinforcement. The goal of the study is to understand the behavior of deficient squat shear walls and determine the effectiveness of using externally bonded CFRP sheets applied to the surfaces of the wall as a retrofitting strategy. The ability of the retrofitting strategy to improve the seismic performance of the wall specimens is evaluated in repair applications in which the specimens are previously subjected to damaging load to simulate an earthquake.

2. Experimental Program

Three shear wall specimens are constructed and tested under reversed cyclic lateral load to understand the behavior of shear walls designed according to obsolete design standards and to determine the effectiveness of using externally bonded CFRP sheets to restore and improve the seismic performance of the specimens in repair applications. The specimens are 1800mm tall, 2750mm wide, and 180mm thick resulting in an overall aspect ratio of 0.65, which is representative of a low-rise squat wall. Each specimen is post-tensioned to the laboratory strong floor and a hydraulic actuator applies the cyclic lateral load 2000mm from the base of the wall panel. Table 1 shows the designation for each specimen, cross-sectional dimensions, the steel and CFRP reinforcement details for each specimen.

Table 1 – Specimen dimensions, reinforcement details and lap splice length

Wall I.D.	**Specimen Type**	**Dimensions (l_w x t_w) (mm)**	**Vertical Steel Reinforcement**	**Horizontal Steel Reinforcement**	**Lap Splice Length**	**Horizontal CFRP Layers**
CW1	Control	2750 x 180	50 - 20M @ 150mm	4 - 10M@500mm	-	-
RW1	Repaired	2750 x 180	50 - 20M @ 150mm	4 - 10M@500mm	-	8
CW2	Control	2750 x 180	50 - 20M @ 150mm	4 - 10M@500mm	$36d_b$	-
RW2	Repaired	2750 x 180	50 - 20M @ 150mm	4 - 10M@500mm	$36d_b$	8
CW3	Control	2750 x 180	50 - 20M @ 150mm	4 - 10M@500mm	$21d_b$	-
RW3	Repaired	2750 x 180	50 - 20M @ 150mm	4 - 10M@500mm	$21d_b$	$8^\dagger$

†Additional confinement reinforcement is provided in the form of CFRP strips for specimen RW3.

Figure 3 shows the steel reinforcement configuration for each specimen in addition to the CFRP retrofitting schemes. Table 2 shows the material properties for the concrete, steel and CFRP reinforcement. Each specimen is detailed with two layers of vertical flexural steel reinforcement and two layers of horizontal steel shear reinforcement. The specimens include three control walls, designated CW1, CW2, and CW3 that are tested without any CFRP reinforcement. Specimen CW1 has no lap splices of the longitudinal steel reinforcement, while specimens CW2 and CW3 have lap splice lengths of 36 and 21 bar diameters (d_b) resulting in lap splice lengths of 720 and 420mm, respectively. The specimens are first tested under cyclic lateral load to simulate earthquake damage and then are repaired using CFRP sheets and tested again, changing the respective designations of the control walls to RW1, RW2, and RW3. Results from the control specimens provides insight into the behavior of existing older deficient shear walls and serve as a baseline for comparison with the CFRP retrofitted specimens. The control wall specimens are expected to exhibit a brittle shear dominant behavior because of their low aspect ratio and the shear deficiencies in their design. The goal of the CFRP retrofitting strategy is to improve the shear capacity of the wall specimens through the addition of horizontally oriented CFRP layers. Using the ACI 440.2R-08 design standard for the application of externally bonded CFRP reinforcement, it is determined that four horizontal CFRP layers are required on each side of the wall to ensure that each specimens capacity against diagonal tension shear failure is greater than the demand associated with the flexural strength of the wall [11]. In doing so, the goal is to prevent premature brittle shear failure and ensure the yielding of vertical flexural steel reinforcing bars closest to the edges of the wall before failure, leading to a higher displacement ductility, energy dissipation capacity, and improved overall seismic performance.

Table 2 – Material properties for the concrete, CFRP and steel reinforcement

Concrete		**CFRP**		**10M Rebar**		**20M Rebar**	
f'_c *(MPa)*	23.0	f_u *(MPa)*	931	f_y *(MPa)*	440	f_y *(MPa)*	480
E_c *(GPa)*	19.2	E_f *(GPa)*	83.8	E_s *(GPa)*	201	E_s *(MPa)*	208
ε'_c *(mm/mm)*	0.0018	ε_{fu} *(mm/mm)*	0.01	ε_{sh} *(mm/mm)*	0.016	ε_{sh} *(mm/mm)*	0.019
f_{test} *(MPa)*	23.5	t_f *(mm/mm)*	1.00	ε_u *(mm/mm)*	0.141	ε_u *(mm/mm)*	0.137

Note: f'_c : 28-day compressive strength; f_{test}: average compressive strength at time of tests; f_y: yield stress; f_u: CFRP ultimate tensile strength; E: elastic modulus; t_f: CFRP laminate thickness; ε'_c: strain at maximum compressive stress; ε_{sh}: strain hardening strain; ε_{fu}: ultimate CFRP strain; ε_u: steel rupture strain;

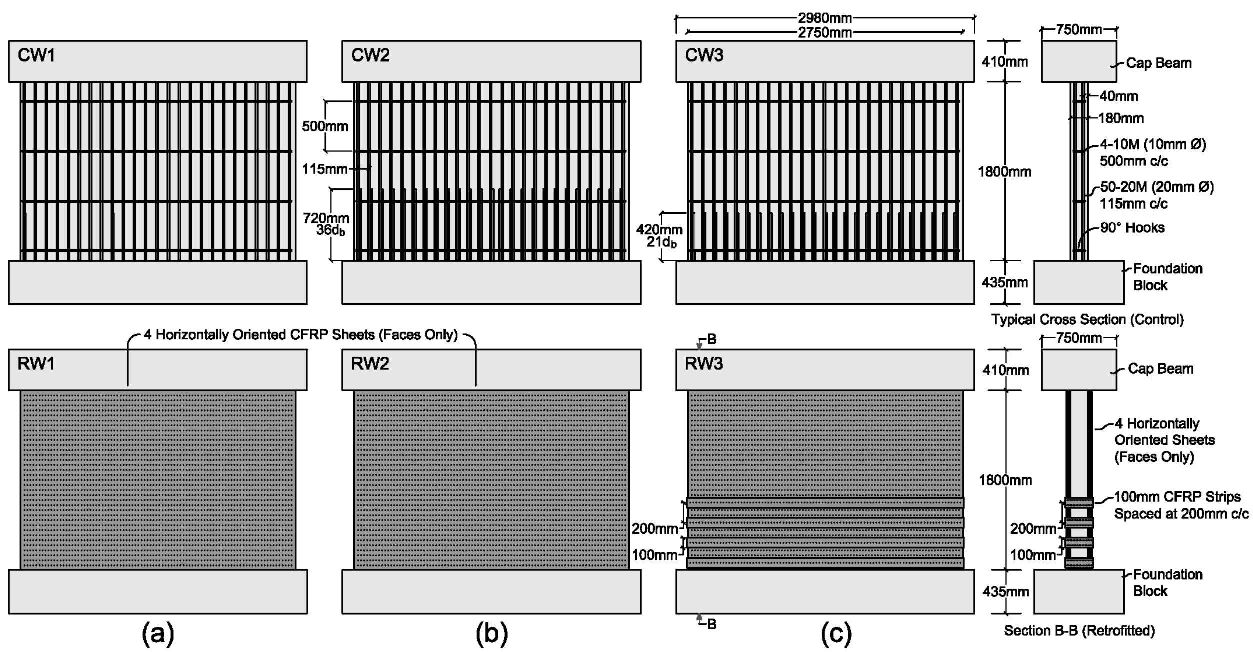
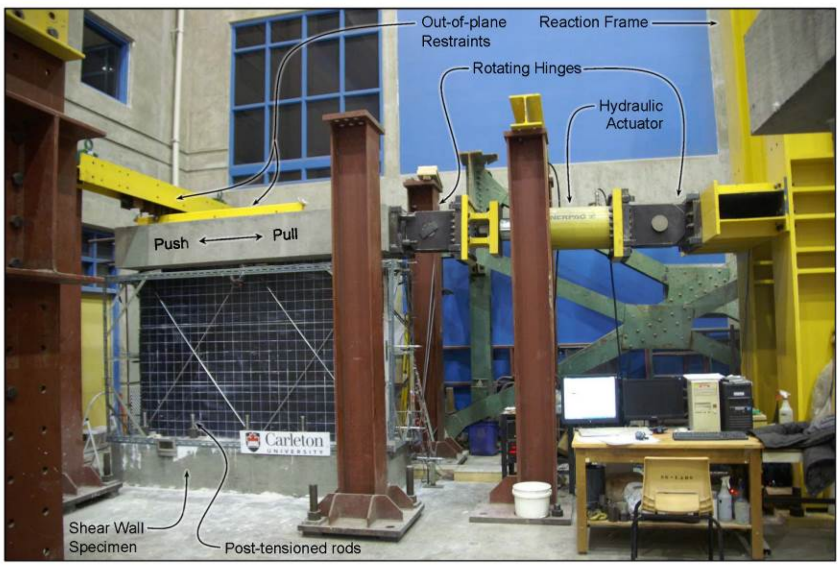


Fig. 3 – Shear wall reinforcement details: (a) CW1/RW1; (b) CW2/RW2; (c) CW3/RW3

Figure 4 shows a typical experimental test configuration for a shear wall specimen in this study. Each specimen is tested in its upright position and post-tensioned to the laboratory strong floor. Out-of-plane deformations of the specimen are prevented by using a lateral restraint system. Two hinges located on either side of the hydraulic actuator ensure that no moments are produced at the top of the specimen such that the wall behaves as a cantilever. The loading protocol consists of applying increasing levels of cyclic lateral load to the top of the specimen in the positive (push) and negative (pull) directions. Each specimen is tested in load control until yielding is achieved in the flexural steel reinforcement. The test is then continued in displacement control by setting target ductility increments ($0.2\mu_\Delta$) up to failure. The highest load carrying capacity achieved during testing is defined as the maximum load. A 20% drop from the maximum load, which is referred to as the ultimate load, defines the failure load and displacement of a test specimen. In this study, axial load is not applied to the specimens. Instrumentation for a typical experimental test includes linear voltage differential transformers (LVDTs) placed at several locations around the specimen to measure top displacement, sliding between the wall panel and the foundation block, shear deformations, rotations and any out-of-plane deformation. Strain gauges on the vertical and horizontal steel reinforcement allows for yielding of any reinforcing bars to be accurately captured during the test. In the specimens with lap splices, additional strain gauges are placed along the length of the two lapped bars to determine the state of bond along the length of the lap splice and determine how effectively stresses are transferred between lapped bars. Strain gauges are placed on the outer layers of CFRP reinforcement to determine the contribution from the CFRP to the lateral load resistance of the wall.



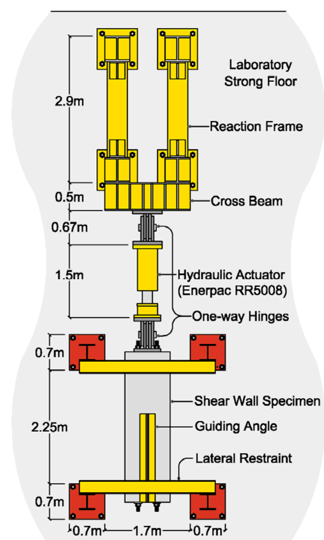


Fig. 4 – Typical experiment test setup (Top View)

3. Experimental Results

The control specimens are tested first to assess the performance of older deficient RC shear walls with or without lap splices. These results will serve as a baseline for later comparison with the retrofitted specimens. Test results for all seven shear wall tests are summarized in Table 3, including the initial stiffness, yield load, maximum load, and ultimate drift achieved. Figure 5 shows the hysteretic response of each specimen compared with its respective control specimen. Note that a full set of experimental results from specimen RW1 could not be achieved because of failure in the foundation of the specimen prior to the specimen reaching its ultimate load carrying capacity. As a result the authors cannot comment on the ductility or the energy dissipation capacity of the wall specimen. However, the specimen achieves some yielding in the flexural steel reinforcement located close to the ends of the shear wall, reaching its flexural strength, before failure of the wall foundation.

Table 3 – Average measured structural response parameters for push/pull cycles

Wall I.D.	Initial Stiffness *(kN/mm)*	Initial Stiffness Relative to CW	Maximum Load *(kN)*	Maximum Load Relative to CW	Ult. Disp. *(mm)*	Ult. Drift *(%)*
CW1†	665	1.00	1160	1.00	4.81	0.27
RW1†	271	0.41	1980	1.70	8.67	0.48
SW1†	606	0.91	2140	1.84	10.5	0.58
CW2†	621	1.00	1390	1.00	8.5	0.47
RW2	200	0.32	1460	1.05	11.2	0.62
CW3	566	1.00	1240	1.00	10.1	0.56
RW3	125	0.22	715	0.58	10.6	0.59

†specimens not tested to ultimate failure, listed values correspond to maximum achieved parameters.

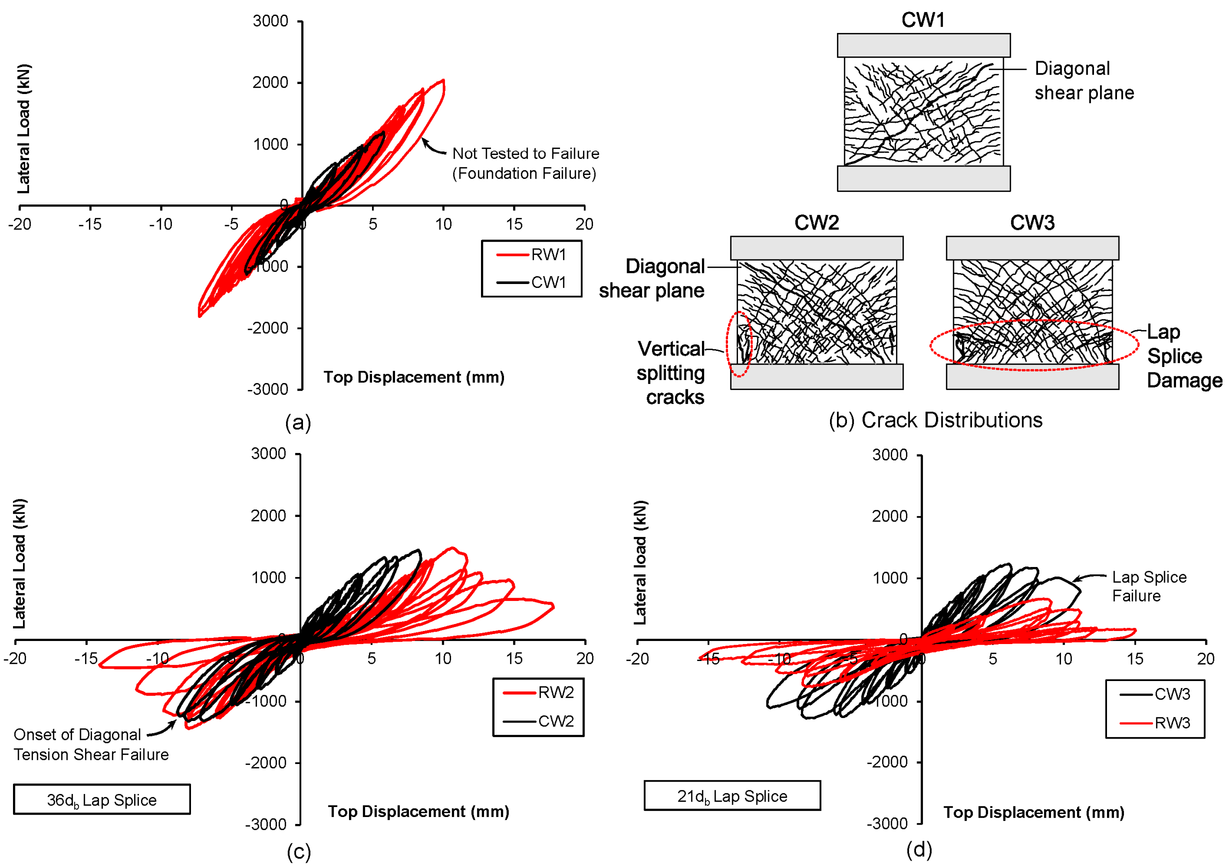


Fig. 5 – Hysteretic response behavior: (a) CW1/RW1; (b) crack distributions; (c) CW2/RW2; (d) CW3/RW3

3.1 Performance of the Control Wall Specimens

Specimen CW1 is tested first and has no lap splices of the vertical flexural steel reinforcement. Figure 6a shows the crack distribution at the end of the test. Initially, diagonal cracks form in the centre of the wall. As the test continues, diagonal cracks dilate and extend from one corner of the specimen to the other. At the maximum load, the horizontal steel reinforcement yields in the centre of the specimen. Past experimental tests conducted by Woods et al. (2016) have shown that an increase in load after yielding of the horizontal steel reinforcement can lead to significant losses in load carrying capacity (up to 40%), leaving the wall with little to no residual shear capacity. As a result, testing of specimen CW1 is stopped at a lateral drift of 0.27% to ensure that a more practical repair scenario could be achieved. The hysteretic response for specimen CW1 (Fig. 5a) shows the minimal ductility and energy dissipation capacity exhibited by the specimen because the flexural steel reinforcement did not yield. These results show the lack of ductility expected from wall specimens with insufficient shear reinforcement because very little yielding in the vertical steel occurs before the horizontal steel yields.

Specimen CW2, which has a $36d_b$ lap splice length, shows very similar behavior to specimen CW1. Diagonal cracks initially form in the centre of the specimen. Under subsequent load reversals the number of diagonal cracks increase, but very few horizontally oriented cracks form when compared with specimen CW1. Horizontal cracks are commonly associate with flexural behavior and thus is an indication that flexural behavior is not as predominant in specimen CW2 because of the lapped flexural steel reinforcement. The first vertical splitting crack along the length of the lap splice is observed at a load of 1040kN and lateral drift of 0.25%. In contrast to specimen CW1, testing is continued past the point of first yielding of the horizontal steel reinforcement in an attempt to attain a more severe damaged state, such as yielding in the flexural steel reinforcement. However, yielding of the flexural steel reinforcement is not observed because of slippage between lapped bars at the ends of the wall, indicated by vertical splitting cracks shown in Fig. 6b. After reaching its maximum load a wide diagonal crack forms from one corner of the specimen to the other at 0.5% lateral drift (Fig. 6b). Upon observation, the test is stopped to prevent brittle diagonal tension shear failure from causing irrecoverable damage to the specimen. The hysteretic response of specimen CW2 (Fig. 5c) shows higher deformability compared to specimen CW1 because of the additional test cycles past the point of first yielding in the horizontal steel reinforcement and the additional energy dissipated from slippage between the lapped bars.

Specimen CW3, which has a $21d_b$ lap splice length shows significant lap splice splitting failure prior to any yielding in the flexural reinforcement or the formation of a wide diagonal shear crack. A horizontal crack at the top of the lap splice is first observed at a load of 500kN and a drift of 0.1%. Vertical splitting cracks along the length of the lap splice at the ends of the wall are observed at a load of 1000kN and 0.23% lateral drift. At 0.56% lateral drift, slippage of the lap splices at the two ends of the wall leads to the formation of a horizontal shear plane at the top of the lap splice, 420mm from the base of the specimen. Failure of the lap splice, indicated

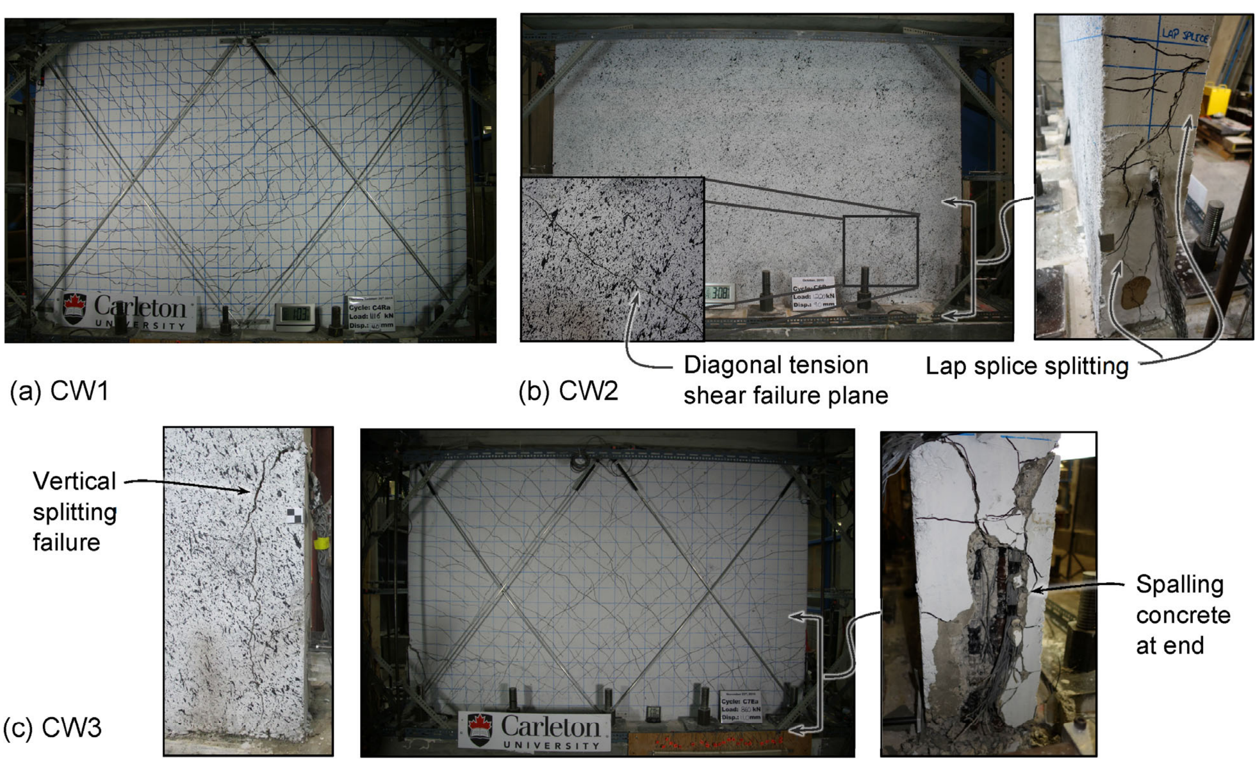


Fig. 6 – Damage to control specimens: (a) CW1; (b) CW2; (c) CW3

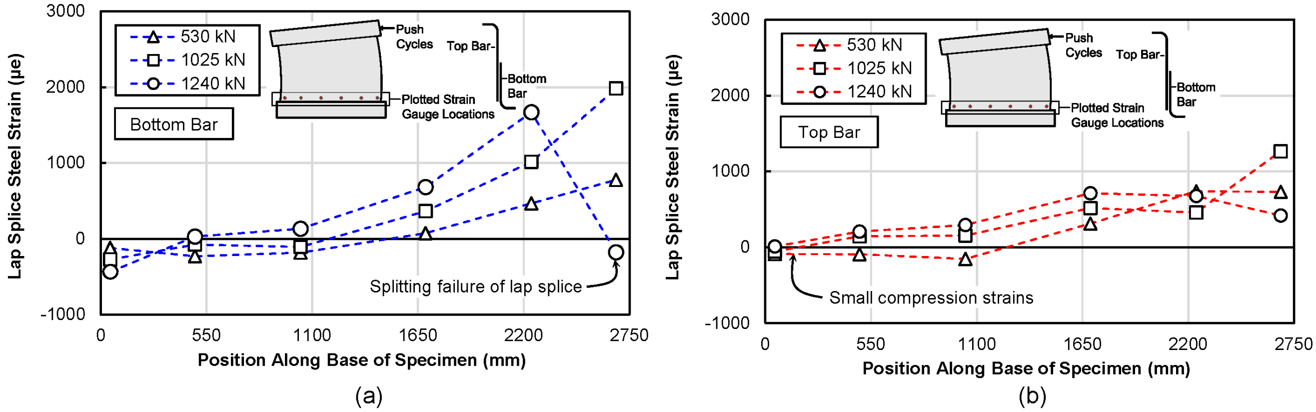


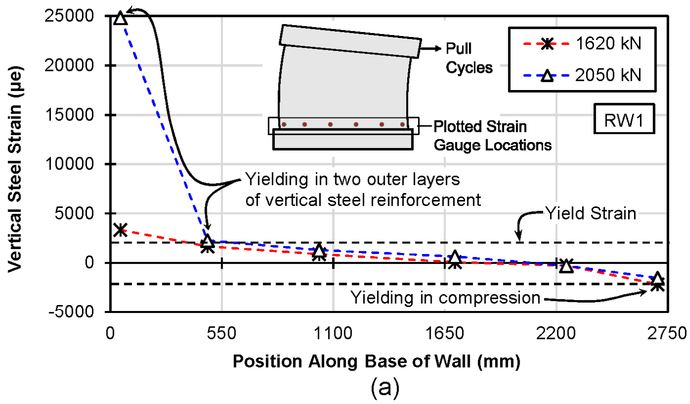
Fig. 7 – Lap splice vertical strain profile for specimen CW3: (a) bottom bar; (b) top bar

on the hysteretic response in Fig. 5c, leads to a drop in lateral load carrying capacity. Figure 6c shows damage to the ends of the specimen during the test, including the vertical splitting cracks and spalling of the concrete around the lap splice. Figure 7 shows the strain distribution along the base of the wall for the top and bottom lapped bars in specimen CW3, respectively. The strain profiles show that at lower levels of lateral load, the strain in the top and bottom bars are very similar, indicating that the lap splice is still able to develop some bond stress. However, the steel flexural reinforcing bars are not able to sustain significant compression strains, suggesting the lap splice length provided is not sufficient to transfer compressive forces between bars. At the maximum load carrying capacity, a significant drop in tensile strain at the edge of the wall occurs as a result of splitting failure of the lap splice before the bars are able to yield in tension. The lack of yielding throughout the vertical steel reinforcement and the tendency for the wall specimens to experience brittle diagonal tension or lap splice splitting failure demonstrates the deficient nature of squat reinforced concrete shear walls with non-ductile reinforcement details. This presents a need for an effective minimally disruptive retrofitting solution for wall deficient squat RC shear walls.

3.2 Performance of the Retrofitted Wall Specimens

To evaluate the effectiveness of the CFRP rehabilitation strategy, the specimens are repaired after being subjected to damaging load. Each specimen is rehabilitated with 4 horizontally oriented layers of unidirectional CFRP laminate (Fig. 3). The CFRP sheets are Tyfo SCH-41 and the epoxy is Tyfo S; material properties for which are shown in Table 2. Before applying the CFRP layers, any loose concrete is removed and areas of spalled concrete are filled with patching mortar. The surface of the concrete wall is then lightly ground to expose the concrete substrate and the CFRP layers are applied in a wet layup process in accordance with ACI 440.2R-08. Because of the insufficient lap splice length in specimen CW3, additional CFRP reinforcement in the form of CFRP strips wrapped around the wall specimen are provided. As illustrated in Fig. 3c, the strips are 100mm wide and are spaced at 200mm on centre over the height of the lap splice. Each specimen is tested under the same cyclic lateral load sequence as the previous control specimens. Structural response parameters for the rehabilitated specimens are shown in Table 3 and a comparison of the hysteretic performance of the rehabilitated walls with their respective control wall is shown in Fig. 5.

In specimen RW1, the application of the CFRP sheets is not able to completely restore the initial stiffness of the wall, but significant improvements in the flexural strength and lateral drift capacity are observed. Specimen RW1 shows very little visible damage and no debonding or separation of the CFRP sheets from the concrete substrate at the maximum load carrying capacity. As shown in Fig. 8a, at the maximum load the first two layers of flexure steel reinforcement in specimen RW1 yield. In addition, the first layer of steel reinforcement in compression also reaches yield at the opposite end of the wall. Figure 8b shows the hysteretic response for the horizontal strain close to the centre of the wall for the horizontal steel and CFRP reinforcement for specimen RW1. Results demonstrate that up to the maximum load carrying capacity, the strains in the steel and CFRP reinforcement remain well below the yield strain (2250μe) of the horizontal steel reinforcement. By



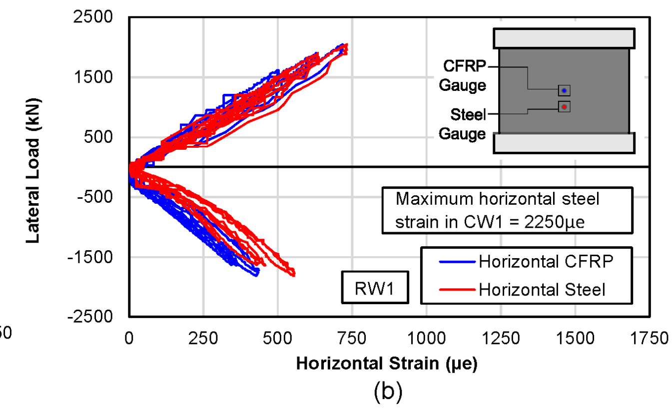
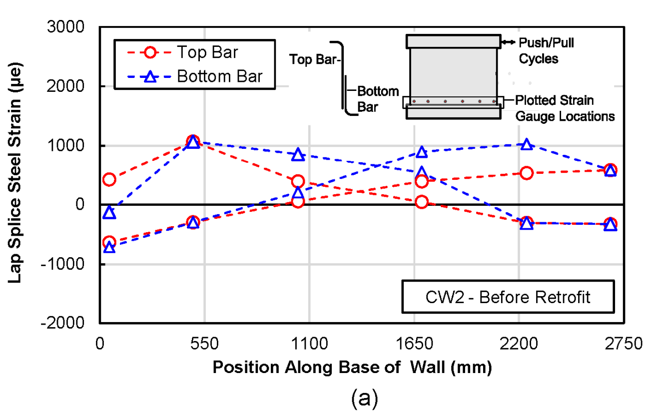


Fig. 8 – Strain distributions in specimen RW1: (a) vertical strain profiles; (b) horizontal strain hysteresis

adding the CFRP sheets, the shear stresses are now transferred across cracks in the concrete by the CFRP sheets and horizontal steel reinforcement together, preventing the horizontal steel reinforcement from yielding and avoiding brittle diagonal tension shear failure. Unfortunately, reaching the ultimate load carrying capacity of specimen RW1 could not be achieved because of premature failure in the wall foundation, which transfers the loads from the wall to the laboratory strong floor. Nonetheless, specimen RW1 exhibits significant strength increases of 170% when compared with the control wall. The test results demonstrate that in a squat wall specimen without lap splices, the application of horizontal CFRP sheets is shown to be capable of significantly improving the lateral load carrying capacity of the wall, eliminating the potential for brittle diagonal tension shear failure and allowing the flexural steel reinforcement at the two ends of the wall to yield.

There is a notable contrast in the performance of the two retrofitted walls with lap splices compared to those without. As shown in the hysteretic response comparison in Fig. 5c and 5d, the application of the horizontal CFRP sheets is successful in restoring the strength of specimen RW2. The retrofitting scheme is not able to restore the initial stiffness in specimens RW2 and RW3. This is likely because the interlocking bond between the separated lapped bars following the initial test load is not restored in an effective manner in the repair process. The initial stiffness of specimen RW2 is only 32% of the initial stiffness of specimen CW2. As shown in Fig. 10a, at an average load of 1020kN, the vertical lap splice cracks formed during the previous test reopen. Specimen RW2 reaches a maximum average load carrying capacity of 1460kN at an average lateral drift of 0.51%. At this stage during the test, splitting cracks along the length of the lap splices at the ends of the wall and a small sliding shear crack along the base of the wall are visible. Under subsequent load reversals, degradation of the strength and stiffness of the wall (Fig. 5c) occurs as the lap splice failure results in a large gap at the base of the wall and a rocking motion as the wall slides back and forth on its foundation. Figure 9a shows the vertical strain profiles at the maximum load in the top and bottom lap spliced bars along the length of the wall for specimen CW2 and RW2, respectively. Comparing the strain profiles in specimen CW2 and RW2



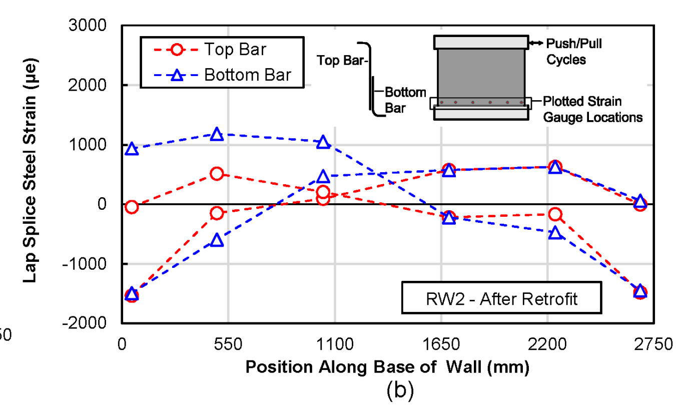


Fig. 9 – Lap splice strain profiles at the maximum load: (a) CW2; (b) RW2

shows that the application of the CFRP sheets is effective in restoring the performance of the lap splice. In addition, the strains on the compressive side are higher in specimen RW2, indicating improved performance due to the presence of the CFRP layers. In specimen RW2, no debonding of the CFRP laminate from the concrete substrate is observed until the specimen enters the post-peak range. As shown in Fig. 10b, the CFRP laminate separates from the concrete and debonds up to the top of the lap splice at the two ends of the wall. Even without providing additional confinement of the boundary elements at the two ends of the wall, the repaired wall specimen has marginally higher deformability and improved energy dissipation capacity compared to the control wall.

In contrast with specimen RW2, the performance of specimen RW3 is not restored to its original capacity as a result of the insufficient lap splice length and the level of damage to the wall during the initial test. Even with the application of the additional CFRP strips wrapped around the wall over the lap splice length, it is not enough to prevent the lap splice from reopening. The initial stiffness of the wall specimen is 22% of the original stiffness of specimen CW3, once again because no effort was made to restore the damaged bond between lapped bars following the initial test. As illustrated in Fig 10c, a horizontal shear plane forms at the top of the lap splice, 420mm from the base of the specimen between the CFRP confining strips. The CFRP sheets have no strength perpendicular to the primary fiber direction, so it is unable to prevent the concrete from splitting at the height of the lap splice. As shown in Fig. 7c, in a similar to specimen RW2, a roughly 15mm gap forms at the base of the wall between the wall panel and the foundation block. During the post-peak cycles, this gap opens and closes as the wall rocks back-and-forth causing significant strength and stiffness degradation that are evident in the hysteretic response. The specimen reaches a maximum load carrying capacity of 715kN and an ultimate lateral drift of 0.6%. The maximum load carrying capacity is 60% of the maximum capacity of specimen CW3.

Experimental results from the specimen with insufficient lap splice length show that more intrusive measures, such as welding of lapped bars, or some means to provide adequate confinement to the lap splice region are required. To add confining pressure to the lap splice, a complete CFRP wrap over the height of the lap splice or the addition of headed bars and a concrete collar could improve the residual capacity of the lap splice.

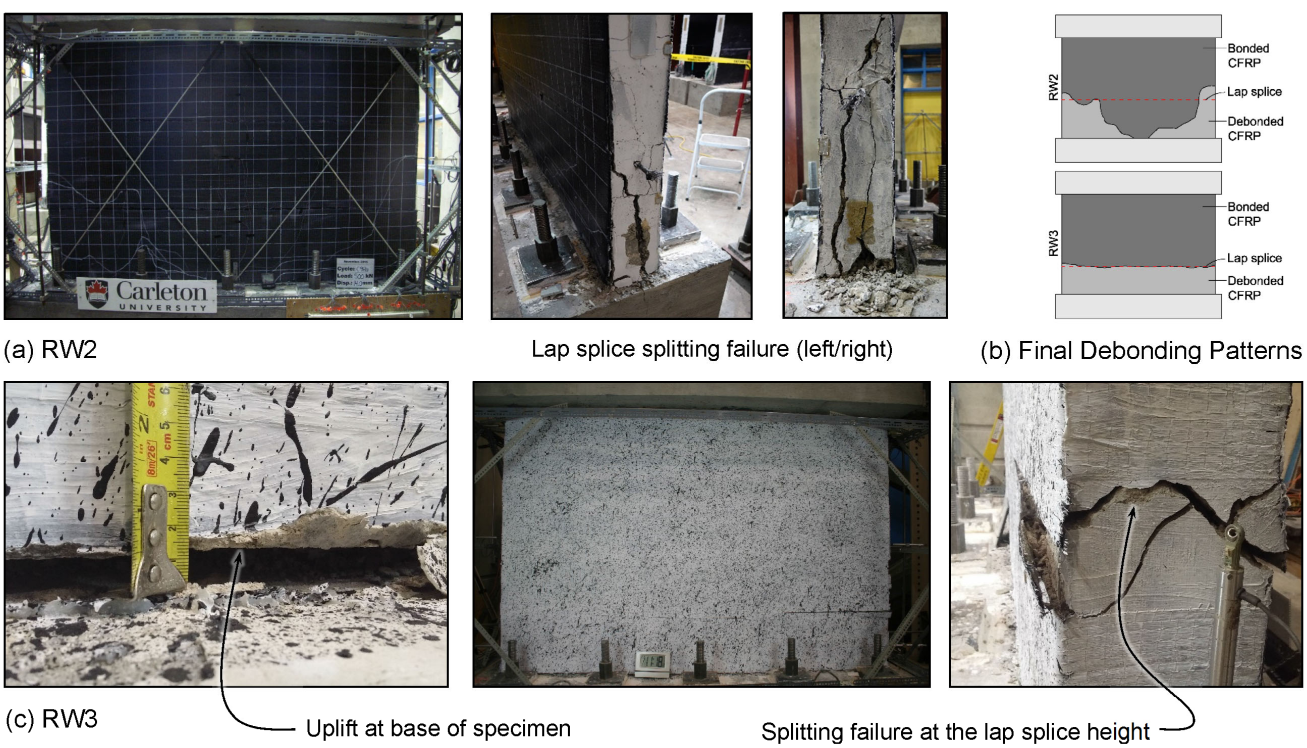


Fig. 10 – Damage to retrofitted wall specimens with lap splices: (a) RW2; (b) debonding pattern; (c) RW3

4. Conclusions

The goal of this study is to gain an understanding of the seismic behavior of low-rise squat RC shear walls with detailing deficiencies associated with older design standards including insufficient shear reinforcement, no concrete confinement at the ends of the wall, and lap splices located within the potential plastic hinge region. The study also investigated the use of externally bonded CFRP reinforcement to restore and improve the seismic performance in repair applications, for shear walls that have been damaged during an earthquake. The goal of the retrofitting strategy is to avoid brittle shear related failures and promote yielding throughout the flexural steel reinforcement leading to better seismic performance. In the wall specimens without CFRP, the formation of brittle diagonal tension shear failure and lap splice splitting failure prior to significant yielding of flexural steel reinforcement is observed. In the repaired wall without lap splices, the retrofitting strategy is shown to be effective in preventing brittle shear failure, and promote yielding of the flexural reinforcement while significantly improving the lateral load carrying capacity of the specimen. The retrofitting strategy is also successful in restoring the in-plane strength of the wall when sufficient lap splice length is provided. When insufficient lap splice length is provided, the retrofitting strategy is not able to restore the strength or stiffness of the specimen. In such cases, a more intrusive retrofitting strategy is required. Future work will focus on additional techniques to improve the ductility of low-rise squat RC shear walls with insufficient lap length.

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