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CYCLIC BEHAVIOUR OF NON-DUCTILE REINFORCED CONCRETE SHEAR WALLS RETROFITTED WITH CFRP SHEETS

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ABSTRACT: This paper presents experimental results on a minimally disruptive retrofitting scheme consisting of externally bonded carbon fibre-reinforced polymer (CFRP) sheets. The retrofitting system is used in non-ductile walls designed according to older less-stringent design standards. The common structural deficiencies in the design of the wall specimens include poor confinement of boundary elements and insufficient shear reinforcement. These deficiencies lead to poor seismic performance characterized by a lack of strength, ductility and energy dissipation capacity. The CFRP retrofitting system is shown to eliminate premature shear failure and enhance the flexural load carrying capacity of the wall specimens. Results on two different anchor systems used to transfer the force from the vertical CFRP sheets to the foundation of the wall are presented. Results demonstrate the potential benefits of using CFRP as a viable retrofitting alternative to improve the seismic performance of non-ductile RC shear walls.

1. Introduction

Research over the past 50 years has resulted in advances in understanding the seismic behaviour and significant improvements in the performance of reinforced concrete (RC) shear wall structures. These advances are reflected in the current design standards for reinforced concrete structures (ACI 318-14; CSA A23.3-14). Despite the advances in seismic design, there is still a large stock of existing shear wall structures designed using older, less-stringent design guidelines (ACI 318-68; CSA A23.3-77). Past experience has shown that these older structures are susceptible to severe damage under moderate to large earthquakes (Saatcioglu et al., 2001; Sezen et al., 2003). The deficiencies in their design that lead to poor seismic performance include insufficient shear reinforcement and poor confinement of the boundary elements. These deficiencies result in a lack of shear strength, ductility, and energy dissipation capacity leading to an undesirable, brittle diagonal tension shear failure. To improve the seismic performance of deficient RC shear wall structures, there are a number of available retrofitting techniques. An innovative, minimally disruptive retrofitting solution is the use of externally-bonded fibre-reinforced polymer (FRP) sheets. The majority of research to date on the seismic retrofit of RC shear walls using CFRP has focused on the use of CFRP jackets to improve shear strength, energy dissipation capacity and confinement of the boundary elements in shear wall specimens (Antoniades et al. 2003; Patterson and Mitchell 2003; Khalil and Ghobarah 2005; Elnady, 2008). However, in many cases, the sides of a RC shear wall are not exposed, and therefore it may be impractical to apply the CFRP sheets around the wall. In addition, some wall specimens may require flexural strengthening in combination with shear strengthening to meet current design standards, something that is not addressed in these studies. In this study, the use of externally bonded FRP sheets to enhance the seismic performance of shear walls designed according to older design standards (ACI 318-68; CSA A23.3-77) is investigated. Five shear wall specimens, expected to exhibit brittle shear behaviour due to insufficient shear reinforcement and poor concrete confinement in the boundary elements are retrofitted with CFRP sheets applied in the vertical and horizontal directions and then cyclically tested to failure. The CFRP sheets are not wrapped around the wall to simulate the most practical installation scenario. The seismic performance of the CFRP retrofitting system is compared with the control wall specimens and evaluated in terms of strength, ductility, and energy dissipation capacity.

2. Experimental Program

2.1. Test Methodology

The shear wall specimens described in this study include five 2/3 scale cantilevered wall specimens designed according to older design standards. More specifically, the walls are designed according to the American ACI 318-68 building code for structural concrete, which is comparable to the Canadian CSA A23.3-77 RC design code. As a result, the walls specimens are designed with several non-ductile details commonly found in old shear wall structures constructed during the 1960s and 1970s. These deficiencies include insufficient shear reinforcement, poor confinement in the boundary elements, and low concrete compressive strength. Table 1 shows the dimensions, height-to-length aspect ratios and reinforcement ratios for the five wall specimens. The aspect ratio for each of the five wall specimens is less than 1.5, and therefore the wall specimens would be considered shear dominant squat walls. The fact that these walls are considered as squat walls, in combination with the deficiencies in their design mean they are expected to fail in a sudden and brittle manner, exhibiting minimal ductility and energy dissipation capacity.

Table 1 – Shear wall dimensions, steel reinforcement ratios, and anchor system type.

Series	Wall I.D.	Type of Specimen	Dimensions (l_w x h_w x t_w)	Aspect Ratio (h_w/l_w)	FRP Anchor System
1	CW1	Control	1.5 x 1.8 x 0.1 m	1.2	-
	RW1	Repaired	1.5 x 1.8 x 0.1 m	1.2	Tube Anchor
	SW1	Strengthened	1.5 x 1.8 x 0.1 m	1.2	Tube Anchor
2	CW2	Control	2.1 x 1.8 x 0.14 m	0.85	-
	RW2	Repaired	2.1 x 1.8 x 0.14 m	0.85	Tube Anchor
	SW2-1	Strengthened	2.1 x 1.8 x 0.14 m	0.85	Tube Anchor
	SW2-2	Strengthened	2.1 x 1.8 x 0.14 m	0.85	FRP Fan Anchors

The wall specimens are detailed with vertical and horizontal steel reinforcement ratios of 3.0% and 0.25% respectively, meeting the minimum requirements outlined by both older design standards (ACI 318-68; CSA A23.3-77), and ensures that the failure of the walls is controlled by shear. The ACI 318-68 design standard had no explicit requirement for the concentration of vertical steel near the boundary elements or additional confinement in these regions, which was not introduced until the publication of the standard in 1971. Figure 1 shows the steel reinforcement layout for a typical wall specimen. The five wall specimens are split into two series (1 and 2) based on the height-to-length aspect ratio (h_w/l_w) of the wall, which are shown in Table 1. The five wall specimens include two control walls (CW1,CW2), which are cyclically tested to failure and then repaired, changing their denomination to RW1 and RW2. The repaired walls provide insight on the ability of the retrofitting system to improve the seismic performance of shear walls that may have experienced some damage during an earthquake and require retrofitting to return the structure to service and meet the current code requirements. The three other specimens are strengthened with CFRP prior to testing with no previous damage, denoted SW1, SW2-1, and SW2-2. The performance of these wall specimens is compared with their respective control wall to evaluate the effectiveness of using externally bonded CFRP sheets as a retrofitting alternative in shear deficient RC shear wall structures. Table 2 shows the concrete, steel, and CFRP material properties. Material properties for the gross laminate section (properties of the cured CFRP system) and an effective laminate thickness are used in the design of the CFRP reinforcing scheme discussed in Section 2.2.

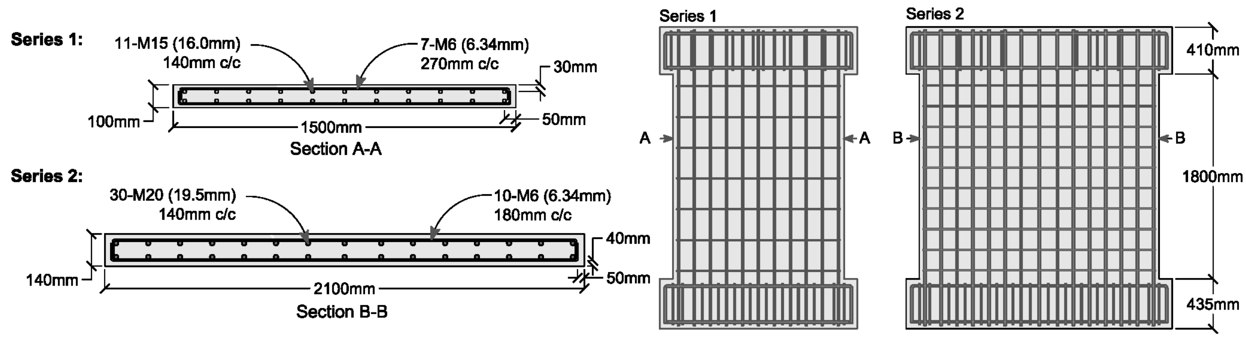

Figure 1 – Typical steel reinforcement detail for W1 and W2 wall specimens.

Table 2 – Concrete, steel, and CFRP material properties.

Concrete and Steel Rebar						CFRP (Tyfo® SCH-41)		
Bar Size	**Prop.**	**Test Value**	**Bar Size**	**Prop.**	**Test Value**	**CFRP State**	**Material Property**	**Test Value**
6M Rebar	f_y	415 MPa	10M Rebar	f_y	439 MPa	Gross Laminate (Epoxy+FRP)	$f_{f,u}$	834 MPa
	ε_y	0.003		ε_y	0.002		E_f	82 GPa
	E_s	140 GPa		E_s	201 GPa		$\varepsilon_{f,u}$	1.0%
Concrete	f'_c	19.1 MPa					t_f *(effective)*	0.33mm

2.2. Design of Shear Wall Test Specimens

The design of the control wall specimens meets the requirements prescribed by the ACI318-68 and CSA A23.3-77 design standards for RC structures. Table 3 shows design flexural and shear strengths for each of the control wall specimens without CFRP reinforcement. A section analysis is used to determine the flexural strength of the plain RC wall specimens using a maximum usable compressive strain in the concrete of 0.0035 according to CSA A23.3-14 (*§10.1.3*). The shear strength against diagonal tension shear failure (V_r) is determined according to the ACI 318-05 (*§21.7*) design code, which is intended for use in the seismic design of RC shear walls. The equation is based on the modified truss analogy approach and takes into account the contributions from the concrete and steel reinforcement, shown in Eq. 1:

$$V_r = \left(\alpha_c \sqrt{f_c^{'}} + \rho_h f_{yh}\right) \cdot A_w \quad (1)$$

where α_c is an aspect ratio coefficient, which is equal to 0.25 for walls with a height-to-length aspect ratio (h_w/l_w) < 1.5; f'_c is the 28-day compressive strength of the concrete; ρ_h is the horizontal steel reinforcement ratio; f_{yh} is the yield stress of the horizontal steel reinforcement; and A_w is the cross-sectional area of the wall. For design purposes, d is taken as $0.8l_w$ for all wall specimens. Results in Table 3 predict the onset of diagonal tension shear failure in both control walls before they reach their expected flexural strength.

One of the main objective of this study is to investigate the ability of the CFRP retrofitting system to increase the in-plane flexural strength of shear dominant RC shear walls designed according to older design standards. To improve the flexural strength of the wall specimens, two layers of vertically oriented CFRP sheets are applied to each wall specimen and anchored to its foundation. To ensure that the wall specimens are capable of reaching their ultimate flexural capacity, shear reinforcement in the form of horizontal CFRP layers are provided to prevent sudden and brittle diagonal tension shear failure. In the design of the CFRP reinforcement, CFRP-concrete debonding must be taken into consideration because debonding between the two materials often governs the failure mechanism (Teng et al., 2002). Debonding between the CFRP and the concrete occurs at areas adjacent cracks in the concrete, which is commonly referred to as intermediate-crack (IC) debonding. When IC debonding occurs, failure of the retrofitted member happens before the material reaches its ultimate tensile capacity, which reduces the capacity of the retrofitted member (Teng et al., 2002; Cruz-et al., 2014). Modern design standards for the application of externally bonded CFRP sheets (ACI 440.2-08; CSA S806-02) take into consideration debonding by limiting the effective strain in the CFRP to the strain level at which cracking occurs. For example, CSA S806-02 (*§11.3.1.1*) provides a strain limit of 0.007 in flexural FRP reinforcement to consider the effects of debonding. A sectional analysis using the limiting strain value in the flexural reinforcement from CSA S806-02 is used to establish the flexural strength of the retrofitted wall specimens, results of which are shown in Table 3.

To enhance the seismic performance of a deficient or damaged shear wall, its capacity against brittle diagonal tension shear failure must be increased beyond its flexural capacity so that the resulting mode of failure is ductile. In a similar manner to the vertical CFRP reinforcement, the strain in the horizontal CFRP is limited to take into consideration the effects of debonding. According to the CSA S806-02 design standard (*§11.3.2.2*), a limiting value of 0.004 is recommended for the design of horizontal CFRP reinforcement in RC shear walls. The capacity against diagonal tension shear failure is the sum of the contributions from the concrete and steel (V_r), plus the additional shear strength provided by the horizontal CFRP layers (V_f). The contribution from the horizontal CFRP reinforcement to the total shear strength of the wall, shown in Eq. 2, is calculated using a method proposed by Seible et al. (1995):

$$V_f = n_s \cdot \left(\varepsilon_f E_f\right) \cdot d t_f \cot\theta \tag{2}$$

where n_s is the number of horizontal FRP plies applied; E_f *is* modulus of elasticity of the CFRP*;* ε_f is the limiting strain for the horizontal CFRP*;* t_f is the thickness of a single CFRP layer; and θ is the angle between the CFRP fibred and the assumed crack, which is conservatively taken as 45º. Although the application of horizontal CFRP layers is successful in increasing the capacity against diagonal tension, it does not contribute to the capacity against sliding shear or diagonal compression shear failure. The capacity of the wall against sliding shear (V_{sl}) is determined according to CSA A23.3-14 (*§11.5.1*), shown in Eq. 3:

$$V_{sl} = \left[c + \mu \cdot \left(\frac{A_v f_{yv}}{t_w d} + \frac{N_u}{t_w l_w} \right) \right] \cdot A_w \leq 0.25 f'_c \, A_w \tag{3}$$

where c and μ are the cohesion and friction parameters, taken as $c=0.5$ *MPa* and $\mu=1.00$; A_v is the area of the vertical reinforcing bars; f_{yv} is the yield stress for the vertical reinforcing steel; and N_u is the axial load on the wall. The capacity against diagonal compression failure is determined using the expression provided in the ACI 318-05 (*§11.10*) design code ($0.83f'^{0.5}_c A_w$). The application of the horizontal CFRP sheets only increases the diagonal tension shear capacity, thus the maximum number of horizontal CFRP sheets that can be applied to the walls is governed by the lesser of the capacity against sliding shear and diagonal compression shear failure. In this study, diagonal compression shear failure is the governing failure mode associated with shear for the retrofitted walls because of the lower concrete strength used in their construction. In all of the retrofitted wall specimens, the additional shear strength provided by the horizontal CFRP sheets ensures that the shear demand associated with the flexural strength of the wall is achieved.

Table 3 – Theoretical design strengths for the wall specimens.

Series	Without CFRP Reinforcement (Plain RC)				With CFRP Reinforcement	
	Flexural Strength (kN)	Diagonal Tension (kN)	Diagonal Comp. (kN)	Sliding Shear (kN)	Flexural Strength (kN)	Diagonal Tension (kN)
1	510	318	557	750	615	1050
2	1420	623	1090	1470	1640	1710

2.3. Anchor Systems

It is commonly recognized that in many cases failure of a CFRP retrofitting system occurs because of debonding between the CFRP sheets and the concrete substrate. CFRP-concrete debonding failures occur prior to the CFRP material reaching its ultimate tensile capacity, preventing the member from reaching its full strength. In an attempt to eliminate or at least minimize CFRP-concrete debonding, an effective anchor must be introduced into the retrofitting system. The purpose of an anchor system is to provide a load transfer mechanism between the FRP sheet and the wall foundation. In this study, two different anchor systems are used to efficiently transfer the load from the CFRP sheet to the foundation of the wall specimen. Table 1 shows type of anchor system implemented for each wall specimen. The tube anchor system is a mechanical anchor system constructed of a cylindrical tube around which the CFRP sheet is wrapped and bolted along the base of the wall. As shown in Fig. 2a, anchor rods are then installed along the length of the wall to concentrically transfer the load carried by the CFRP sheet to the foundation of the wall. Detailed design and optimization of the tube anchor system is discussed in more detail by Woods (2014).

The second anchor system implemented in the SW2-2 wall specimen is a series of carbon and glass FRP fan anchors placed along the base and sides of the shear wall respectively. Fan anchors, which are sometimes referred to as spike anchors, are fabricated from strands of bundled composite fibres. CFRP anchors, measuring 510mm in length are embedded into the foundation of the wall specimen to transfer the load carried by the vertical CFRP sheets to the concrete foundation. Smaller 150mm glass fibre-reinforced polymer (GFRP) anchors placed along the edges of the wall prevent premature debonding of the horizontal CFRP sheets. Fig. 2b shows the dimensions and layout of the CFRP and GFRP fan anchors. Results on the performance of the anchor systems are presented in Section 3.4.

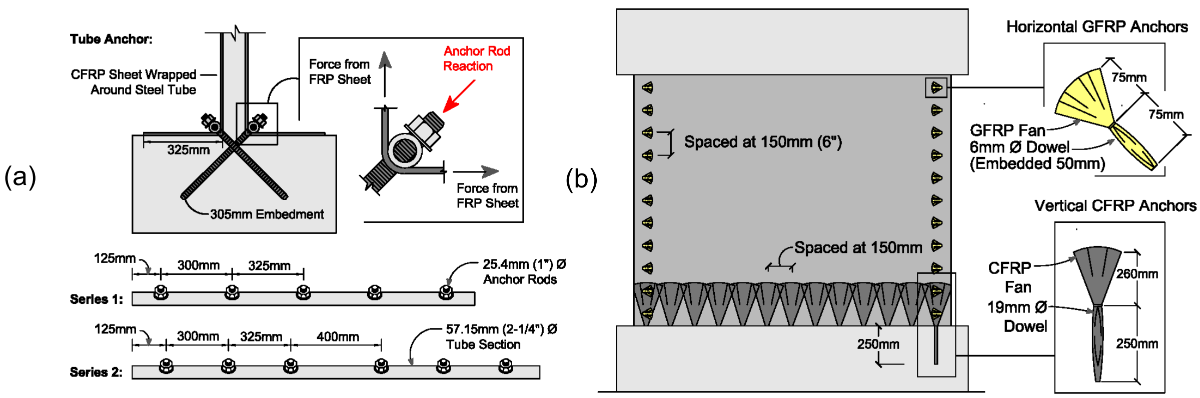

Figure 2 – (a) Tube anchor system design details; (b) CFRP/GFRP fan anchor system layout.

2.4. Cyclic Load Sequence and Test Setup

The wall specimens are tested under a quasi-static reversed cyclic lateral load sequence to simulate the effects of earthquake ground motions on a RC shear wall. Fig. 3 shows a typical test-setup. The wall specimens are secured to the laboratory strong floor and a hydraulic actuator applied to cyclic lateral load to the top of the wall specimen. Due to limitation on laboratory equipment, axial load is not applied to the test specimens. The wall specimens are first tested in load control by applying two successive cycles at 25%, 50%, 75%, and 100% of the estimated yield load. The test is then continued in displacement control by increasing the target displacement ductility up to failure. At each target displacement ductility level, the load cycle is repeated twice to study any softening effects the recursive load cycles might have on the strength and stiffness of the wall specimen. Out-of plane deformations of the wall specimen are minimized using a lateral restrain frame.

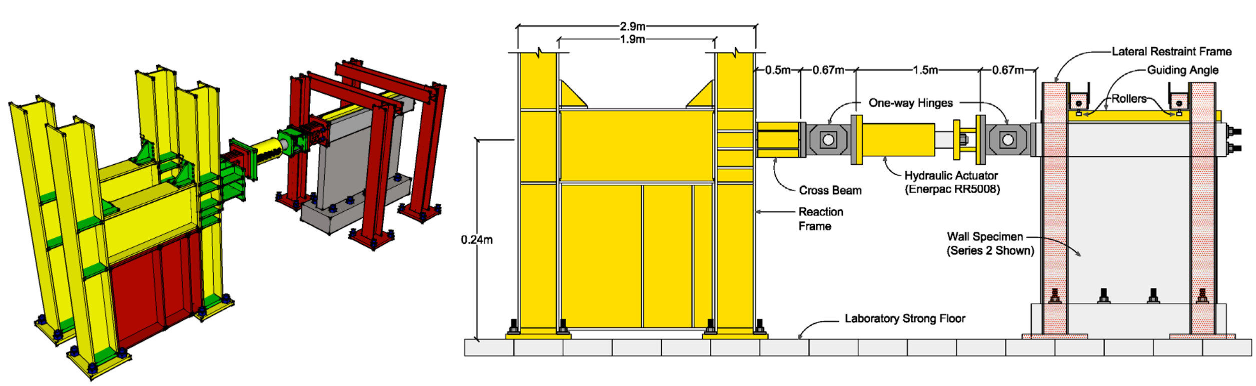

Figure 3 – Typical laboratory test setup (W2 wall specimen shown).

3. Experimental Results

3.1. Control Wall Specimens (CW1/CW2)

The lateral load versus the top displacement hysteretic response and crack distribution pattern for the two control wall specimens is shown in Fig. 4. Diagonal cracking in the concrete is first observed at an average drift of +/-0.05% in the centre of the wall specimen. Table 4 shows important structural response parameters for all of the wall specimens. Yielding of the extreme layer of vertical reinforcement occurs at an average lateral load of +/-242kN and 765kN for series 1 and series 2 control walls respectively. As predicted, failure of the CW1 and CW2 wall specimens occurs in a sudden and brittle manner in diagonal tension shear at lateral drifts of 0.74% and 0.54%, and lateral loads of 341kN and 1020kN respectively. These results correlate well with the design strengths shown in Table 3, which predict the onset of diagonal tension well before the wall reaches its flexural capacity. The poor seismic performance of the control wall specimens is reflected in their hysteretic response, which shows minimal energy dissipation capacity and ductility prior to failure. The crack distributions in Fig. 4 show minimal flexural cracking and concrete crushing prior to failure. The displacement ductility (μ_Δ) of the wall specimens is determined by dividing the equivalent yield displacement (Δ_y) by the ultimate displacement (Δ_u) of the wall. The equivalent yield load (P_y) and displacement are determined using bilinear idealization of the force-displacement envelope. The maximum displacement occurs when the load drops to 80% of the ultimate load carrying capacity measured during testing.The average displacement ductility of the control wall specimens is 1.20 and 1.25 for specimen CW1 and CW2 respectively, showing the lack of ductility in walls detailed with deficiencies associated with older design standards.

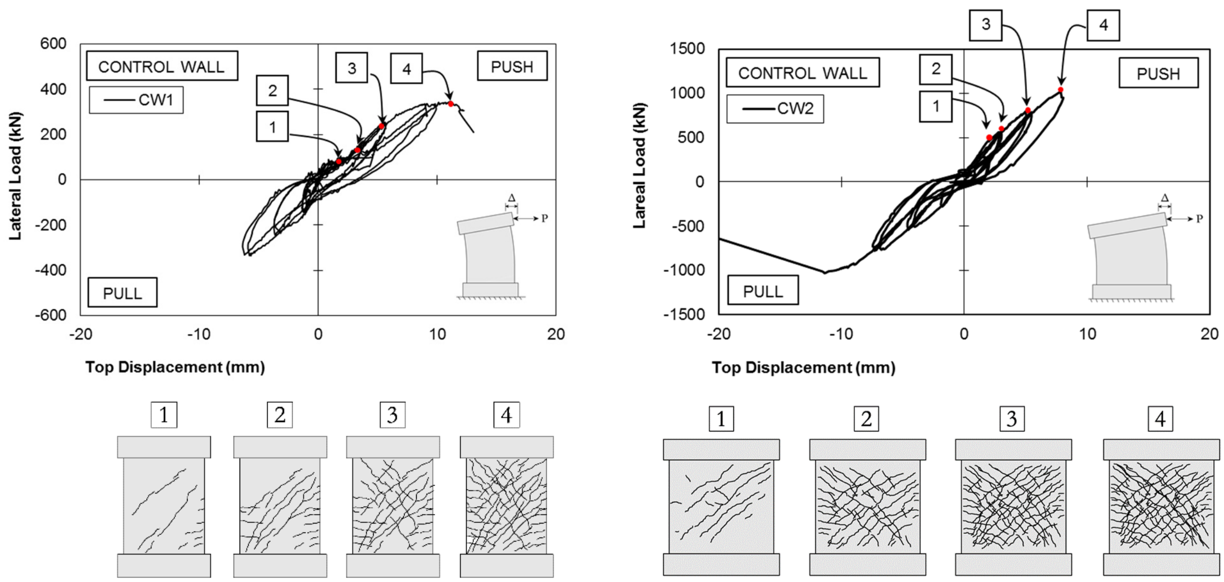

Figure 4 – Hysteretic response and crack progression for specimens CW1 and CW2.

Table 4 – Structural response parameters for each wall specimen.

Series	Wall I.D.	Yield Load (P_y) (kN)	Post-Yield Stiffness (kN/mm)	Max Load (kN)	Equivalent Yield Disp. (Δ_y) (mm)	Failure Disp. (Δ_u) (mm)	Max Drift Ratio (%)	Disp. Ductility (μ_Δ)
1	CW1	242	22.4	341	11.0	13.2	0.74	1.20
	RW1	372	40.6	610	8.58	26.6	1.48	3.10
	SW1	376	51.4	633	7.10	29.1	1.62	4.10
2	CW2	765	75.0	1020	9.12	11.4	0.54	1.25
	RW2	777	72.8	975	9.15	12.9	0.72	1.41
	SW2-1	909	112	1505	7.21	21.0	1.17	2.91
	SW2-2	835	133	1405	7.39	19.0	1.05	2.57

3.2. Repaired Wall Specimens (RW1/RW2)

After testing the control walls, they are repaired using a combination of epoxy resin, patching mortar, and composite CFRP sheets. In contrast to previous studies (Antoniades et al., 2003; Paterson and Mitchell, 2003; Khalil and Ghobarah, 2005; Elnady, 2008), the CFRP sheets are not wrapped around the wall in a jacket to account for the fact that in some cases in the field, the sides of a RC shear wall may not be exposed. This is an attempt to make the CFRP retrofitting system more practical and less disruptive when compared to the application of a CFRP jacket, while still improving the seismic performance of the wall.

The repaired walls are tested again to failure and the force versus top displacement response for both wall specimens and CFRP-concrete debonding pattern is shown in Fig. 5. Debonding of the CFRP from the concrete substrate occurs at drift ratios ranging from +/-0.2-0.38% along the diagonal of the wall in areas which experienced extensive cracking during the previous test. At higher levels of drift (+/-0.57-0.88%), debonding initiates at the toes of the wall, which is attributed the presence of flexural cracks and concrete crushing at the base of the wall. During subsequent load cycles, concrete crushing and FRP-concrete debonding spreads to the interior of the wall, indicating a shift in behaviour to a more flexural mode of failure. Rupture of the vertical CFRP in tension occurs at the ultimate load carrying capacities of 610kN and 975kN for specimens RW1 and RW2 respectively. Although both wall specimens exhibit signs of flexural behaviour, the CFRP retrofitting system is unable to dramatically improve the seismic performance of the wall specimens because of the level of damage to the walls during the previous test. Specimens RW1 and RW2 fail at lateral drifts of 1.48% and 0.72%, resulting in an overall displacement ductility ratio of 3.10 and 1.41 respectively. Comparison of the hysteretic behaviour of each wall shows the ductility and energy dissipation capacity of the wall specimens is not increased dramatically when compared with the control walls, however, the retrofitting system is capable of restoring or slightly exceeding the performance level of the walls when compared to their original state. This shows the potential for using CFRP as a repair strategy for severely damaged walls or as a retrofitting solution to increase the capacity and performance in walls with a lesser amount of pre-existing damage.

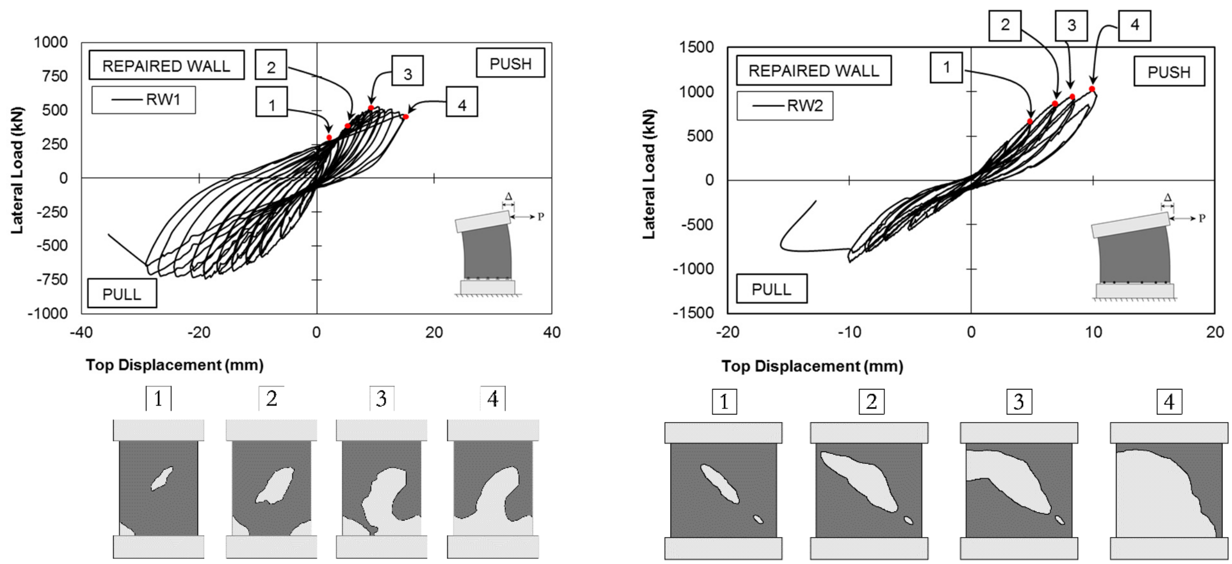

Figure 5 – Hysteretic response and debonding progression for specimens RW1 and RW2.

3.3. Strengthened Wall Specimens (SW1/SW2-1/SW2-2)

The CFRP sheets are applied to the strengthened wall specimens in the same manner as the repaired walls. The lateral load versus top displacement, debonding progression and crack distribution are shown in Fig. 6. The initial cracks in the concrete occur at an average drift of +/-0.04% in the form of horizontal flexural cracks. The presence of flexural cracking prior to extensive diagonal cracking indicates a shift in behaviour from predominantly shear to a more flexurally dominant behaviour. Yielding of the extreme steel reinforcement occurs at an average load of +/-376kN and 872kN for the SW1 and SW2 specimens respectively, which corresponds to an average increase in the yield load by 24% for the strengthened wall specimens. CFRP-concrete debonding in the strengthened wall specimens first occurs at the base of the wall at average drift ratios ranging from +/-0.82-0.88%, which is higher than the repaired walls (+/-0.2-0.38%). This clearly demonstrates the influence or limitation the pre-existing damage has on the response of the repaired wall specimens. CFRP-concrete debonding at the base of the wall is once again attributed to the opening of large cracks in the concrete. The increase in shear and flexural strength provided by the horizontal and vertical CFRP sheets increases the lateral load carrying capacity of the walls to an average of +/-633kN and 1455kN for the SW1 and SW2-1/SW2-2 wall specimens respectively, which is a significant improvement compared to the control walls. At the ultimate load carrying capacity, yielding throughout all of the reinforcing bars along the base of each wall specimen is achieved, indicating that the wall specimens are able to reach their flexural load carrying capacity. Experimental results correlate well with design strengths in Table 3, which predicts the walls will reach their flexural strength prior to failing in shear. The strengthened wall specimen's exhibit significant improvements in ductility when compared to the control walls, achieving displacement ductility ratios ranging from 2.5-4.0. The strengthened wall specimens also exhibit higher energy dissipation capacity, identified by wide loops in the hysteretic response. In strengthening applications, the retrofitting system is shown to be able to increase the flexural strength, ductility, and energy dissipation capacity while preventing premature diagonal tension shear failure.

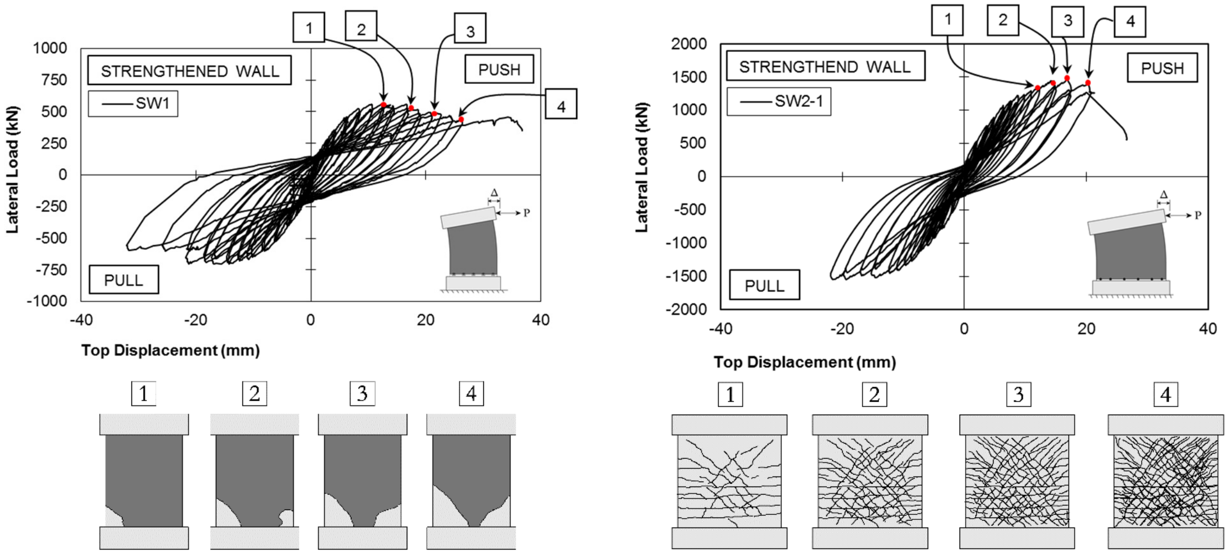

Figure 6 – Hysteretic response, debonding/crack progression for specimens SW1, SW2-1.

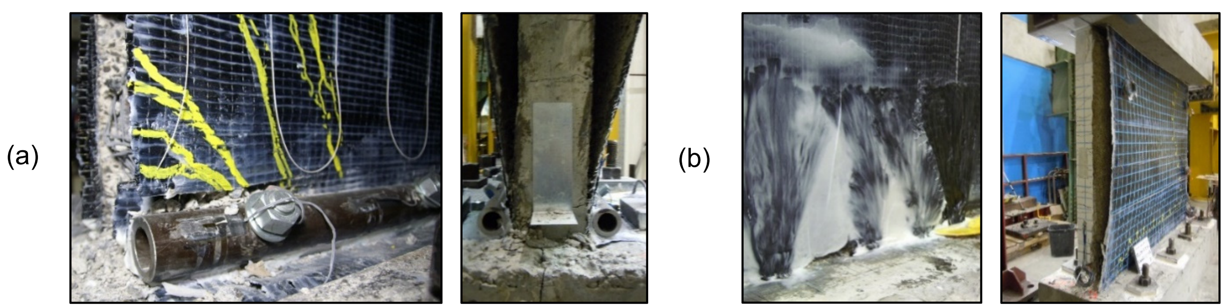

Figure 7 – Anchor system performance: (a) tube anchor; (b) CFRP/GFRP fan anchors.

3.4. Anchor System Performance

Results from the tests show that both anchor systems perform well in transferring the load from the CFRP laminate to the foundation of the shear wall specimen. The tube anchor system prevents debonding of the vertical CFRP sheet and allows the CFRP to reach its ultimate tensile capacity, identified by tearing of the vertical CFRP at the ultimate load carrying capacity of the wall specimens, shown in Fig. 7a. The optimized procedure used to design the tube anchor system, discussed by Woods (2014) is shown to perform well in improving the efficiency of the tube anchor system while still ensuring optimal performance. The vertical

CFRP anchors, shown in Fig. 7b, are also effective in preventing premature debonding and allow the wall specimen to reach its flexural strength. The application of horizontal GFRP anchors along the edges of the wall allow the wall to reach higher levels of lateral drift before debonding of the CFRP layers from the concrete substrate when compared with the other strengthened wall specimens.

3.5. Displacement Contributions from Flexure, Shear and Sliding Shear

To quantify the shift in behaviour of the retrofitted wall specimens from a shear dominant to a more flexurally dominant behaviour, the total tip displacement of each wall is broken down into contributions from flexure, shear and sliding shear. Results are shown at the cracking, yield, and ultimate load in Fig. 8. Because of the shear dominant nature of the squat wall specimens ($h_w/l_w < 1.5$), the total displacement response of the control walls is dominated by shear deformation. Specimen CW2 has a lower height-to-length aspect ratio ($h_w/l_w=0.85$) compared to specimen CW1 ($h_w/l_w=1.2$), thus a larger percentage of the total displacement comes from shear deformation. For the repaired wall specimens, shear deformation makes up over 70% of the total tip displacement. This is attributed to the reopening of the diagonal failure plane formed during the previous test. When comparing the strengthened and control wall specimens, a significant shift to a more flexurally dominant behaviour is evident, with flexural deformation making up a larger portion of the total displacement. This indicates a shift to a more flexurally dominant behaviour when compared with the control wall, as the strengthened wall specimens experience more steel reinforcement yielding and concrete crushing prior to failure. This shift in behaviour is also confirmed by the increase in ductility and energy dissipation capacity observed in the hysteretic response behaviour for all of the strengthened walls.

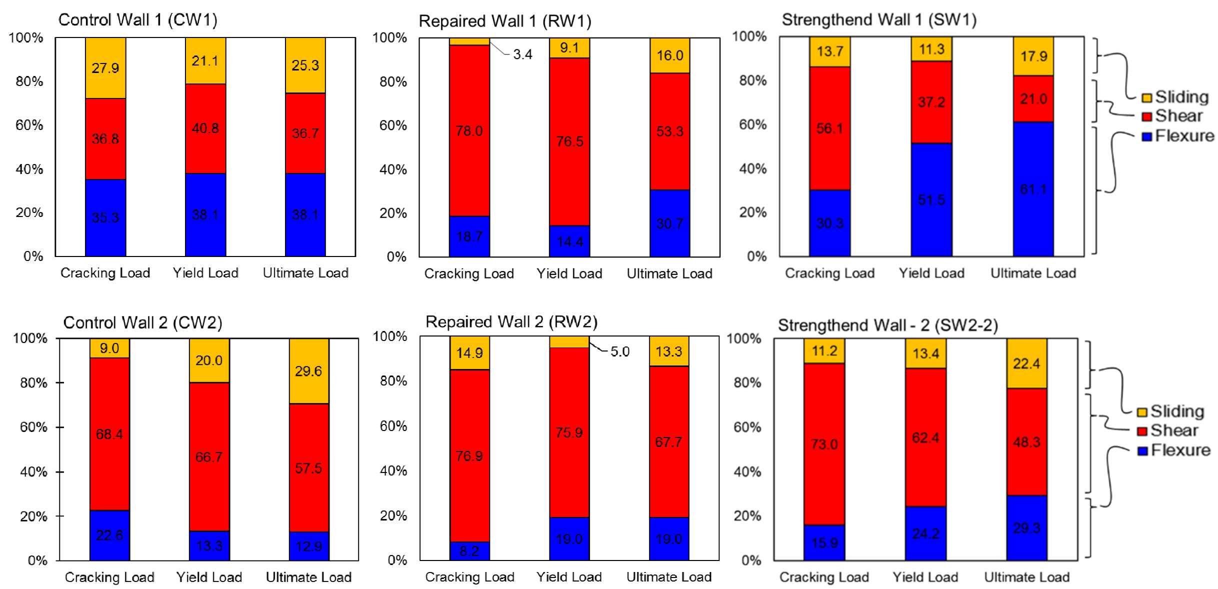

Figure 8 – Displacement contributions from flexure, shear and sliding shear.

4. Conclusion

This paper presents results of a study on the performance of squat RC shear walls designed according to older design standards, in particular ACI 318-68, which is comparable to the CSA A23.3-77 design standard for RC structures. Results of the study demonstrate that walls designed according to older design standards, particularly those detailed with insufficient shear reinforcement and poor confinement of the boundary elements are susceptible to sudden and brittle diagonal tension shear failure with little to no ductility or energy dissipation capacity. To address this issue, an innovative CFRP retrofitting system is investigated. Results show that by applying externally bonded CFRP sheets in combination with an effective anchor system the retrofitting system is capable of preventing premature shear failure and enhancing the seismic response of deficient shear wall specimens. The use of CFRP sheets in vertical and horizontal directions performs well in increasing ductility and energy dissipation capacity in strengthening applications. Even in severely damaged walls, the retrofitting system is capable of restoring the wall to its original state, which shows the potential for application of the retrofitting system in walls with lower levels of pre-existing damage. Experimental results show that the shear strength of a wall specimen can be significantly increased even without wrapping the CFRP sheet around the wall. Both anchors systems are shown to be effective in delaying premature CFRP-concrete debonding and allowing the CFRP sheet to approach its ultimate tensile capacity. An effective anchor system is once again shown to be a crucial component of the CFRP retrofitting system.

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